THE BROADWAY SERIES OF ENGINEERING HANDBOOKS

BRIDGE FOUNDATIONS

W. BURNSIDE

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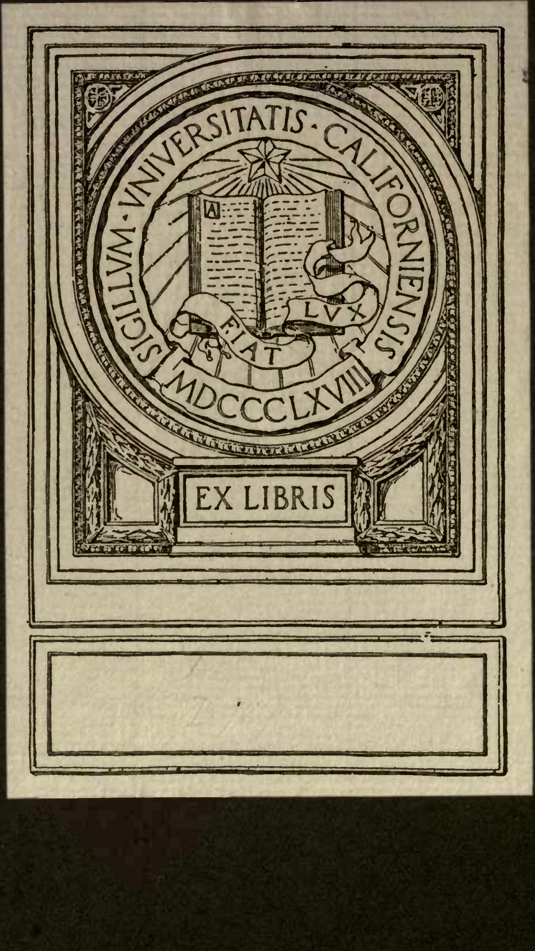
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BRIDGE FOUNDATIONS

THE BROADWAY SERIES OF ENGINEERING HANDBOOKS
VOLUME XIX

BRIDGE FOUNDATIONS

BY

WM. BURNSIDE, Assoc.M.Inst.C.E.

WITH THIRTY-TWO DIAGRAMS

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PREFACE

The book is intended only as an introduction to the subject of bridge foundations, and has been written in the scant leisure of ordinary business. There is room for a careful and comprehensive work on bridge foundations. The best books known to me are American, and they leave something to be desired by British engineers. Much might be done towards standardizing the practice in design, as regards both the determination of loads and the allowable pressures on beds. There is not even a uniform nomenclature for bed materials.

My practical experience of bridge foundations was gained under my former chiefs, the late Sir William Arrol, LL.D., and Mr. A. S. Biggart, M.Inst.C.E., and I am glad to make the acknowledgment. I have to thank other friends for particular information, and I would mention Mr. Adam Hunter, M.Inst.C.E.; Mr. Arthur J. Knowles, M.Inst.C.E., Cairo; Mr. W. A. Cashman, M.Inst.C.E.; Mr. Gunni Jeppesen, Chicago, and Mr. John H. Taylor, Assoc.M.Inst.C.E., Buenos Ayres.

My thanks are due, also, to Messrs. Sir Douglas Fox & Partners for drawings and information relating to Chiromo Bridge; to Mr.

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H. C. V. Möller for the Copenhagen bridges; and to Mr. G. W. Rear, Bridge Engineer, Southern Pacific Co., California, for particulars of concrete caissons.

I have to acknowledge, with thanks, information received from the Barrowfield Iron Works, Ltd., the Foundation Company, New York, and the steel piling Firms mentioned in the text.

Mr. Paul M. Plews has kindly compiled some of the tables and in other ways extended his help, and many of the illustrations have been made from drawings prepared for me by Mr. John Wallace, Jun.

WM. BURNSIDE.

Glasgow,
October, 1915.

CONTENTS

CHAPTER I

PAGES
INTRODUCTION 1-4

CHAPTER II

THE FOUNDATION BED 5-11
Supporting Power—Depth of Foundations.

CHAPTER III

ABUTMENT FOUNDATIONS 12-16

CHAPTER IV

PIER FOUNDATIONS 17-23
Ordinary Dead Loads—Special Dead Loads—Live Loads—Wind—Current, Ice and Floating Bodies—Braking Effects.

CHAPTER V

PIER FOUNDATIONS (*continued*) 24-26
Skin Friction—Buoyancy, etc.

CHAPTER VI

PILED FOUNDATIONS 27-33

CHAPTER VII

DAMS 34-50

CHAPTER VIII

SCREW PILES AND SCREW CYLINDERS . . . 51-58

CHAPTER IX

PAGES
WELL FOUNDATIONS 59-66

CHAPTER X

STRANDED CAISSONS 67-75

CHAPTER XI

OPEN CAISSONS 76-83

CHAPTER XII

PNEUMATIC CAISSONS 84-94

CHAPTER XIII

PNEUMATIC CAISSONS (*continued*) 95-101

CHAPTER XIV

THE EFFECTS OF COMPRESSED AIR 102-113

CHAPTER XV

CONCLUSION 114-121

TABLES I-IX 123-134

I. Loads on Foundations.
II. Skin Friction.
III. Properties of Materials.
IV. Pile-bearing Formulæ.
V. Steel Sheet Piling.
VI. Quantities and Costs of Pier, American River Bridge.
VII. Decompression Periods—Drs. Hill and M'Leod.
VIII. Stage Decompression—Dr. Haldane.
IX. Dr. Snell's Table.

INDEX 137-139

CHAPTER I.

INTRODUCTION.

The title is taken here to refer only to foundations for abutments and piers of bridges crossing rivers or other waters: that is to say, to foundations which are wholly or in part subaqueous. It is of these we think usually when we speak of bridge foundations. Foundations for overline bridges, underline bridges over roads, and viaducts across valleys and ravines, to which ordinary building methods are applicable, are not considered. The foundation is defined as that part of the substructure which is under ground or under water as the case may be, low water level being taken in the case of tidal waters, and ordinary water level in other cases. In other words, the foundation is the part that is hidden and out of sight. A foundation may be said, broadly, to consist of two parts—the foundation structure and the foundation bed, called simply the foundation and the bed. The former transmits the loads, or active forces, to the latter whose function is reactive or supporting.

Bridge foundations sustain the concentrated loads arising from the weight of the structure itself, the traffic carried, and any external forces, such as wind, acting upon it. The foundations are taken into account in the general design of a bridge; and while on the one hand the conditions affecting them may be not unusual, on the other, they may be such as to influence considerably and even wholly determine the type of bridge and its linear dimensions. Take, for example, the case of an ordinary girder bridge on a regular bed. If there are no special requirements as to openings, generally a series of equal spans would be adopted, and for economical design the ratio of cost of substructure, inclusive of piers, to cost of superstructure, exclusive of roadway or permanent way materials, should be kept about 1 : 1, as this is held generally to give the best result for simple girder bridges. In ordinary circumstances this ratio holds for a span of about 170 feet. Should the foundations be difficult, and therefore relatively expensive, the span length must be increased and the number of piers reduced; whereas, if the foundations are easy the span may be shortened with advantage. The Hawkesbury Bridge, New South Wales, had until recently the deepest foundations known, being 155 feet maximum depth below low water, and the spans are 416 feet c/c, which is getting near the economical limit for independent girders. The Forth Bridge is an instance in which the foundations, being so far apart, limited the bridge to a suspension or a cantilever type. A suspension design was adopted at first, but subsequently superseded in favour of the existing cantilever bridge. The bridge over the Zambesi at Victoria Falls is another instance of foundations, which are on the sides of the gorge, practically determining the design.

The difficulty of foundations increases with their depth, and also, to a less extent, with the depth of water over the river bed.

There is not much latitude in design of abutment foundations, unless they are pier abutments, but there is more scope with pier foundations. The general form of these may be arrived at by a process of elimination, somewhat as follows. Skeleton foundations, such as piles, are not suitable for very heavy concentrated loads in any circumstances, and are unsuitable for much lighter loads in even comparatively shallow waters, because of their lack of lateral stiffness and the practical difficulties of stiffening them against flexure. Mass foundations may be in series or continuous, according to the loads to be carried, but if subject to heavy lateral forces, such as floods carrying floating masses, the continuous form only is admissible. Sometimes there is an advantage on a sloping bed in using several units in the length of a pier, so that the units may be stopped at different depths. In a long pier it may be necessary to found with two or more units, in order to limit the size of caissons.

The construction and details of a foundation depend largely on the materials to be used, and the method to be adopted in getting the foundation to its bed. Personal experience and familiarity, also, may create a prejudice in favour of one type more than another. The durability of a foundation should not be overlooked.

The different kinds of foundations in common use, and the methods by which they are placed in position, are described in the following pages. The conditions suited to each kind are noted, and where necessary and possible the principles of design are indicated. Next to nothing, however, is said with regard to cost, and in reference to plant, as the consideration of these would lead too far.

Civil engineering is not an exact science, and the design of bridge foundations is perhaps one of the least exact departments of it. Too many unknowns enter into the study, and personal judgment has to be largely used. Theories and formulæ are only of limited application. The difficulties of foundations, however, do not lie in the design, but in the actual execution; and the solving and mastering of these difficulties are learned much better from practical experience than from books.

CHAPTER II.

THE FOUNDATION BED.

A FOUNDATION bed (1) must be capable of supporting safely the net load imposed on it; and (2) it must be at such a depth as to be beyond disturbance by weather influences, by draining, or by scour; or, alternatively, must be protected against these.

1. **Supporting Power.**—In most foundations the principal reaction is obtained from the bed, and therefore its supporting power is an important factor. The supporting power of a bed may be inferred from earth or mineral bores put down at the site of the foundation; such bores, also, are a necessary preliminary to design; it may be found from actual experiment, by means of trial cylinders or otherwise, before the foundation is designed; or, in the case of beds laid bare during construction, direct tests may be made. All of these courses may be followed in some cases, and it is customary to test important foundations after completion by actual loading to an extent 20 to 50 per cent in excess of the greatest combination of loads possible on the foundation. Important foundations are those in which subsequent settlement would have serious effects, as, for instance, in arch bridges, continuous girder bridges, and in most types of movable bridges.

Slight uniform settlement in foundations of simple girder bridges is not usually objectionable. Irregular settlement is obviated by adopting the same unit load on adjacent foundations on uniform material, and in varying materials by, if possible, proportioning the load to the bearing capacity. Settlement due to the crushing and not the compression of the material is inadmissible.

A bore is made, starting either from dry ground or from a submerged surface, by driving down a steel or iron tube 4 inches in diameter or more, at the same time that the boring tool is worked down. The tools vary in form for different materials—augers for clay, chisels for rock and gravel, and others—and the borings are brought up by a valvular tool called a valve. In soft material the valve alone is used. A record or journal is kept of the strata passed through, and the depths at which changes occur, the strata being judged from the borings and from the resistance offered to the tools. Sections are drawn from the journal, and these are the preliminary data for design. Too great reliance should not be placed on bores. In the first place, the bores may have struck a fault or irregularity which does not indicate the general formation; and, in the second place, a perfectly representative bore may be wrongly interpreted even by an experienced borer. It need scarcely be said that only competent and experienced borers should be employed. Even a series of bores, as is proved by actual experience, does not always eliminate errors. There is the further consideration that there is no standard nomenclature and value for strata or materials bored. Two borers may mean different materials under the same description, or may describe different materials in the same way; and, also, the meaning conveyed to the engineer may be erroneous. Bores, as has been said, are a necessary preliminary, but the design of a foundation should be such as to admit of its being stopped short or carried farther, or modified if required.

The net load which should be placed upon any material, judged from the bores, by actual examination of the excavation or of the bed, or in other ways, is greatly a matter of judgment based upon experience. Loads which have been placed upon foundations of the same kind vary within fairly wide limits, and foundations rarely fail from settlement only. It should be said, however, that it is not possible always to judge from published records if figures given are gross or net values. Table I gives the limits of safe bearing capacities for reasonable practice—at ordinary depths, except for rocks. At greater depths, say 35 feet and over, these values might be increased for firm materials, because of their greater density, and with alluvium and other soft materials there is a certain balancing action of the surrounding material which increases with the depth and may be taken into account.

The load which should be placed on foundations for which actual tests have been made is also a matter of judgment, and may be decided by the amount of settlement admissible, these settlements with their corresponding loads being noted during the tests. The testing of an exposed bed may be done by means of pig iron or other loading, or by hydraulic pressure applied by jacks. The test load should be added in unit increments, say of 10 cwts. per square foot, and a pause made between each addition until settlement has ceased. If the loads and settlements, or sinkages, are plotted on a chart,

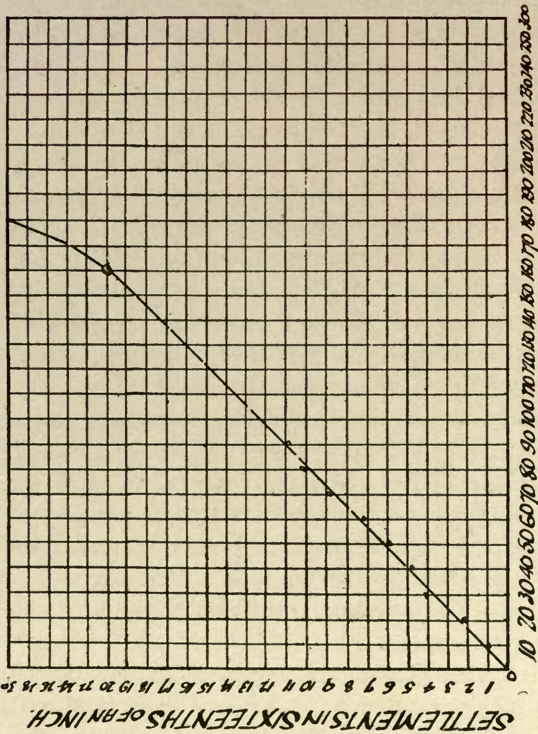


FIG. 1.—Load—Settlement curve.

a load-settlement curve will be obtained, as Fig. 1. This curve up to a certain point may be approximately a straight line of regular inclination, the settlements being nearly proportionate to the loads. Beyond this point, which may be called the point of break-down, settlements increase in an increasingly rapid ratio, and the inclination of the curve steepens. The advantage of loading and recording in this way is that a more or less definite point may be ascertained beyond which judgment need not be exercised, and below which the allowable load may be fixed by the admissible amount of settlement, as stated. This should have reference to the nature of the total load and its source, and also to the reliableness and comprehensiveness of its computation. A close idea of the character of a stratum may be obtained by an experienced man from the use of a shovel, pick, or steel bar driven down. Pig iron and old rails are usually employed as loading for trial cylinders, and for the ultimate test loads of finished piers. The load which can be applied in this way is bulky in proportion to the cylinder or pier, and great care must be exercised in properly distributing, bonding and plumbing the test load, especially in the case of deep and small cylinders.

Foundations are ordinarily made in sand, gravel, clay, in mixtures of these, or on rock. Sand and gravel are accumulations derived from the mechanical disintegration and removal of rocks. They necessarily vary much in composition and quality—say from quick—or running—sand to compact beds of gravel and shingle. Sand, either wet or dry, provided it is prevented from lateral movement, makes an excellent foundation, as does gravel, and mixed sand and gravel. Usually the more homogeneous the material is the better it is. Clay is formed from the decomposition of rocks, and like sand and gravel varies much in composition and quality. Wet clays are undesirable, and the value rises from this quality, through firm yellow clay, up to solid blue clays, and clay with boulders. Firm sand and bound gravel or hard pan are almost incompressible, gravel is more compressible, and with clay compression practically varies with the load. Sound rocks will carry any load which can be brought upon them by a foundation structure. They must be, however, sound, free from fissures, clay beds and other defects; and if overlying beds of softer materials should be of sufficient thickness considered as a beam to sustain the loads. Care must be taken not to found on shelving rocks or rocks with sloping beds which may be liable to slide. Rankine allows the safe bearing powers of rocks to be one-eighth of the ultimate crushing load.

2. **Depth of Foundations.**—As before stated, unless special protection is provided, the foundation bed must be at such a depth as to remain undisturbed. An exposed bed will be beyond the disintegrating influence of frost or drought if it is not less than 4 or 5 feet under the surface; and draining of permeable soil must be prevented. These considerations apply practically only to abutments. Pier foundations are almost invariably what are known as deep foundations ranging from, say, 30 feet to as much as 170 feet below water surface, and the minimum depth may have to be fixed from consideration of scour. Scour may be present in any river, and is, of course, aggravated by obstructions such as piers; and the tendency to scour may be increased or decreased by changes of the bed. A sandy channel may gradually change its position, silting up in one place and scouring out in another. Rivers of this

kind are common in India, and the practice there in the course of many years has been changed gradually from long bridges with numerous comparatively shallow foundations, crossing all the possible courses of a river, to much shorter bridges with deep foundations crossing a trained and directed bed. Scour may be periodic, due to floods or other causes; it will generally be lessened by increasing the navigable channel; it is necessary to allow at least for an equivalent cross-section of water-way being eroded to that already existing before piers were constructed; in navigable rivers the ultimate dredging level must be taken into account. It will be understood from these remarks that each case must be decided by a study of local conditions, circumstances, and probabilities, and that no general guidance can be given. The piers of many bridges to-day are periodically protected by stone pitching cast round them, either because proper consideration was not given to their depth originally, or because conditions have changed beyond anticipation. Permanent protection in the form of enclosing piling, cement grouting of the ground, and in other ways has been resorted to.

CHAPTER III.

ABUTMENT FOUNDATIONS.

An abutment is a combined pier and retaining wall. In addition to supporting its own weight and the dead and live loads of and on the structure, it has to resist the overturning effect of the retained earth. These loads are transferred to the foundation block, which is to be considered here. Take only the case of a simple girder bridge. The equivalent resultant of all the loads on the foundation will vary in position, direction and magnitude with the position of the live load; that is, it will fluctuate within certain limits. If, for example, the live load is immediately behind the abutment it will act as a surcharge on the retained earth and increase the overturning moment, whereas if it is wholly on the span it will increase the vertical component of the resultant.

It is a condition of stability that there should be no tension on any horizontal joint in the abutment or foundation, and for this condition to be fulfilled the resultant must never pass outside the middle third of the breadth. Fig. 2 is a section of an abutment, and BC the middle third of its breadth. The resultant load should be within BC. To demonstrate this, suppose that there is no stress at D and that it is a maximum at A: then a triangle AED would represent the total pressure on AD, and from geometry the resultant of this pressure acts through B, the edge of the middle third. It follows from similar reasoning that if the resultant passed to the left of B there would be negative pressure, or tension, at D. This principle can be applied to any imaginery joint of the foundation block, but it is obvious that if the resultant is much inclined and the block for any reason very thick it would need to be very broad to satisfy the condition. Practically we know that at depths varying from 5 to 10 feet, according as it is firm or open, the embedding soil and foundation act together, so that this aspect need not be considered beyond the depths stated.

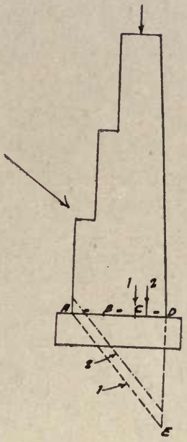


FIG. 2.

The distribution of the pressure on the block and bed varies with the position of the resultant. Consider a unit length (1 foot = b) of the abutment, the direction of the resultant being vertical. If it acts through the c.g. (i.e. the centre line) of the block the pressure on the bed will be uniform. If W = load, A (or breadth $l \times 1$) = area, and P = intensity or unit pressure—then

$$P = \frac{W}{A} \quad . \quad . \quad . \quad (1)$$

If, however, the load is eccentric there will be bending, and the pressure will be increased at one edge and decreased at the other. In Fig. 3, ABDC is the foundation block, and the rectangle CKLD represents the pressure due to central load W. Suppose the load W_1 to act at a distance x from W, and let P_1 = pressure at C, and P_2 = pressure at D. Then

$$P_1 = P + \frac{6\,W_1\,x}{l^2} \quad . \quad . \quad . \quad (2)$$

and

$$P_2 = P - \frac{6\,W_1\,x}{l^2} \quad . \quad . \quad . \quad (3)$$

and the figure CMND is a diagram of the pressure.

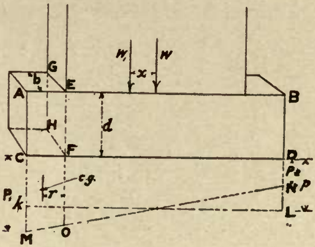


FIG. 3.

Example. W or W_1 = 20 tons. l = 10 feet. x = 12 inches.

$$P = \tfrac{20}{10} = 2 \text{ tons per sq. ft.}$$

$$P_1 = 2 + \frac{6 \times 20 \times 1}{10 \times 10} = 3{\cdot}2 \text{ tons per sq. ft.}$$

$$P_2 = 2 - \frac{6 \times 20 \times 1}{10 \times 10} = 0{\cdot}8 \text{ do.}$$

It will be seen that the average pressure is constant. The maximum edge pressure must be limited so that the bed may not yield and tend to cause failure by tilting. It is assumed that the block is of such a quality as to transfer the pressure regularly, which is a reasonable assumption. The case of a vertical resultant only has been taken, but an inclined resultant can be resolved at any horizontal plane into its vertical and horizontal components. The vertical component would be treated as above, and the horizontal component would produce a horizontal shear or tend to slide the block on its bed.

Consider now the stresses produced in the foundation block, Fig. 3, by load W_1. Disregarding its own weight and the weight of superimposed soil, maximum bending will occur at EF producing maximum tension (f_t) at F, and maximum compression (f_c) at E. The amount of the bending moment will be the pressure on the footing CF multiplied by the distance r of the c.g. of CMOF from FO : or

$$\text{BM} = \frac{\text{CM} + \text{FO}}{2} \times \text{CF} \times r \quad . \quad . \quad (4)$$

It will be met by the moment of resistance MR of the section GEFH.

On the assumption that the neutral axis of the section passes through its c.g.,

$$\text{MR} = \tfrac{1}{6}\, b\, d^2 f \quad . \quad . \quad . \quad (5)$$

where b (as before) = breadth in inches, d = depth in inches, f = extreme fibre stress of the material in pounds.

From (4) and (5)

$$f = \frac{3 \times (\text{CM} + \text{FO lb.}) \times \text{CF} \times r}{bd^2} \quad . \quad (6)$$

in pounds per square inch.

Shear on GEFH $= f$

$$f_s = \frac{\text{CM} + \text{FO}}{2} \times \text{CF} \div bd \quad . \quad . \quad . \quad (7)$$

in pounds per square inch.

For plain concrete f in compression (f_c) should not exceed 300 lb. per square inch, in tension (f_t) 50 lb., while f_s should not exceed 60 lb.

Example. Take the previous weight W_1 and length. Let the footing = 18 inches, and the depth 30 inches. Find extreme tension.

$$f_t = \frac{3 \times 6720 \times 1{\cdot}5 \times 9{\cdot}6 \text{ in.}}{12 \times 30 \times 30}$$

$$= 27 \text{ lb. per square inch.}$$

and

$$f_s = \frac{6720 \times 1{\cdot}5}{12 \times 30}$$

$$= 28 \text{ lb. per square inch.}$$

The shearing strength would probably in most cases determine the minimum thickness of the foundation block. The block may rest directly on the excavated bed, or on piles as described in a later chapter.

CHAPTER IV.

PIER FOUNDATIONS.

THE loads which act or may act on a pier and its foundation are these:—

1. Dead loads from weights of foundation, pier, and superstructure.

2. Dead loads from water and earth, if any, superimposed on the foundation.

3. Live loads on superstructure.

4. Wind loads on exposed surfaces.

5. Current, ice, and floating bodies, mainly on submerged surfaces.

6. Lateral forces due to the application of brakes, to friction and temperature.

The reactions which are or may be present in the foundation are these:—

7. The supporting power of the foundation bed.

8. The friction of the soil on the embedded surface.

9. Buoyancy of a foundation structure on a permeable bed; or, the weight of displaced water and earth in the case of a foundation on an impermeable bed.

These loads and reactions will be considered now in detail, but first it may be said that they are not all of equal importance, and, also, that they vary in importance in different cases, although with perhaps the exceptions of (2), (5), and (6) they all exist in any foundation. Which of them are practically worth considering in any case depends on the conditions. For example, wind forces on structures of no great height, especially where the ordinary dead and live loads are comparatively heavy, might be disregarded. Skin friction, again, depends directly on the depth of embedded surface and in shallow foundations may be taken as non-existent. In many foundations the dead and live loads are simply equated to the safe supporting power of the bed; or, in order to allow for other factors not investigated, the value taken for the bed may be increased or decreased in the judgment of the designer.

1. **Ordinary Dead Loads.**—The dead loads due to weights of foundation structure, piers, and superstructure can be estimated closely without difficulty. They may have to be taken approximately in the first instance, but corrected figures should be used in the final stages of design. These loads act vertically downwards in simple girder bridges and in some other types. With arch, cantilever, and suspension bridges the resultant of the loads from superstructure should preferably be vertical but may be inclined.

2. **Special Dead Loads.**—Cylinder foundations, particularly, are often splayed out at the foot, so that the areas of the shaft and base are proportionate to the bearing capacities of the pier and bed respectively. Whether the weight of the ring of soil and water resting on the projecting base is worth considering can be decided by inspection.

3. **Live Loads.**—The live loads due to trains, vehicular, and pedestrian traffic must be taken at their probable maximum. If the live load is large in comparison with the combined dead loads, and also in comparison with the actual mass of the substructure, its impulsive effect (or impact) on the bed should be allowed for either directly by adding a percentage to the live load, or in determining the allowable unit pressure on the foundation bed. In small span railway bridges with high-speed trains, the allowance may be as much as 50 per cent of the live load; whereas on a heavy long span railway bridge a 5 per cent addition may be sufficient. Authorities differ as to the percentage to be allowed for impact. J. A. L. Waddell's formula for railway bridges is

$$I = \frac{400}{L + 500}$$

where I = impact, and L = length of span in feet. Let S = live load, and s = live load plus impact. Then

$$s = S + SI.$$

The live load on a girder bridge acts vertically through the bearings, so that it will fluctuate between the bearings of the two supported spans. The resultant thrust from arches, etc., will vary with the position of the moving load.

4. **Wind.**—Wind is assumed to act horizontally in either direction on all exposed surfaces and at a slight angle to the axis of the bridge. The worst condition with regard to the superstructure may be when it is covered by the moving load. On an un-

loaded span the pressure should be assumed to be 56 lb. per square foot on the surface of windward and on half the surface of the leeward girders; and on

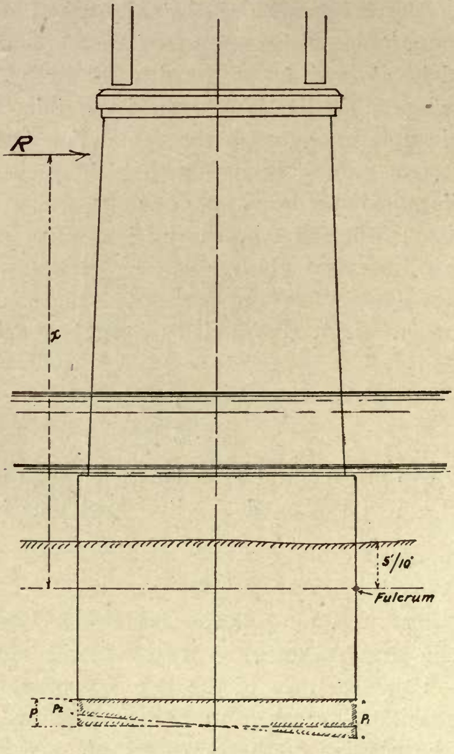


FIG. 4.

a loaded span at 30 lb. per square foot on the same surfaces, except for the part of the leeward girder sheltered by the train or other moving load, plus the area of the moving load. The Board of Trade requires 56 lb. wind to be taken in all conditions for railway bridges. The wind on the pier may be taken at 56 lb. per square foot on the end surface. The resultant of all the wind loads can be found by figuring out the moments about any plane, or by means of a polar diagram. It will have a horizontal shearing effect on the foundation, and an overturning effect. The arm of its moment as regards the pressure on the bed will act at a distance 5 to 10 feet below the surface of the ground, according to the nature of the ground; and it will vary the intensity of the pressure of the vertical loads from a minimum at the windward edge, or toe, to a maximum at the leeward, the average pressure being unaltered. Fig. 4 shows in outline the elevation of a continuous pier and its foundation, which illustrates the foregoing remarks. R = resultant wind load, x = arm of load and the hatched diagrams represent the intensity of pressures with and without wind.

5. **Current, Ice, and Floating Bodies.**—These forces are not usually of sufficient importance in this country to be considered specially, although piers have been damaged and sometimes destroyed by vessels out of control. But in American rivers, for example, they are of very great importance. The pressure of a current against a pier foundation depends on the velocity of the current and on the shape of the foundation. Weisbach's formula is:—

$$P = SWK \frac{V}{2g},$$

where P = pressure in pounds per square foot.
S = exposed surface in square feet.
W = weight of a cubic foot of water.
K = a coefficient depending on the ratio of width to length of foundation.
V = velocity of current in feet per second.
g = acceleration of gravity.

K varies from 1·33 for square figures to 1·46 for rectangles whose length is three times their width; for cylinders K is 0·73. Another formula is given below.

Assuming that ice crushes past a foundation, the pressure will be equal to the crushing strength of the ice multiplied by the area crushed. In computing the stability of the piers of the St. Louis Bridge, the crushing strength was taken at 40 tons per square foot. The area crushed will be the greatest width of the pier multiplied by the thickness of the ice.

The impact from a floating body can be calculated from the weight of the body and the velocity of the current.

Let I = impact, W = weight of colliding body, V = velocity in feet per second, and g = the effect of gravity. Then $I = \frac{Wv^2}{2g}$.

The shearing and overturning effects of these three forces may be considerable, and will in many cases necessitate the use of a continuous pier and foundation, and not a row or series of cylindrical or rectangular shafts, or an open type.

6. **Braking Effects.**—The application of the

brakes to a moving train will have a lateral thrust on the piers to which the superstructure is fixed. The free piers of a continuous girder bridge, or a pier which only supports sliding bearings, will be unaffected except by friction, usually assumed to be 25 per cent of load. The maximum effect will be produced by a quick-moving train suddenly brought to rest, and the magnitude of it will depend on the weight of the train, the speed at which it is travelling, and the distance through which it is brought up. A safe rule is to allow $\frac{1}{5}$ the weight of the train as a horizontal force, which must be taken up by the fixed piers through the superstructure.

Formula for pressure of water in motion used in example, p. 26 :—

V = velocity of current in feet per second.

P = pressure in pounds per square foot on a plane normal to the current.

Then $P = 1{\cdot}85\ V^2$ for salt water; and relative value for P on cylindrical surface is equal to from 0·54 to 0·57 that on normal surface. In example 0·5 has been taken. The resultant pressure may be taken as acting at one-third the depth of the water from the surface.

CHAPTER V.

PIER FOUNDATIONS (*Continued*).

7. The supporting power of the foundation bed has been discussed in Chapter II, and further consideration is unnecessary.

8. **Skin Friction.**—The friction of the surrounding soil on the embedded surface of a foundation, or skin friction, depends on the soil, the depth embedded, and the nature and extent of the surface. It will depend also whether a particular surface is true or not, that is flat, or twisted and buckled—a rectangular caisson, for example, might have surfaces anything but flat; and on the foundation being plumb or out of plumb when sunk. It may range from 1 to 5 cwts. per square foot. Table II gives a list of values obtained by experiment during construction of actual foundations. It does not appear to be customary to allow any definite value for skin friction in design, it being held vaguely as something to the good. In deep foundations, however, it is generally a very real supporting force. It should not be assumed to act for, say, the first 10 feet in depth of ground.

9. **Buoyancy, etc.**—If the foundation is bedded on a permeable bottom where there is water, it will be water borne, and the total value of the support will be the product of the area of the base and the pres-

sure due to the head of water. One pound per square inch for every $2\frac{1}{4}$ feet of head is the usual value allowed. If, on the contrary, the foundation rests on a sealed bottom, the weight of the overlying water and earth removed may be considered as a supporting force, since these produced an initial load of which the bed has been relieved. It will be seen, therefore, that the two cases are practically equivalent. There are floating foundations which support the total loads, or the greater part of them.

The tendency of the loads acting on a pier foundation is to cause failure by crushing of the materials or bed; by shearing on planes generally horizontal, or by sliding one portion over another, or the whole on the bed; and by overturning. These are met by the reactions, and by the resistance of the structure itself. And for security the reactions, supposed quiescent, must exceed the loads considerably.

Vertical forces acting through the centre of gravity of the foundation produce a direct compressive stress only; if all the vertical forces have an eccentric resultant there will be bending in addition, as indicated in Chapter II; inclined forces have a vertical component producing direct stress or with bending stresses in addition, and a horizontal component tending to shearing or overturning.

The general dimensions of a pier of whatever type will usually be determined by the width of the bridge supported, the length being practically settled from this, and the thickness or width fixed from the loads. Whether the foundation should be solid and continuous, or composed of separate units, depends on the loads, and the nature of bed, depth of water, and character of other physical features.

Calculations for a Twin-Cylinder Pier.—The design of a usual type of pier, consisting of twin cylinders braced together, is shown in Fig. 5. This example shows the application of most of the principles and statements mentioned in this and the preceding chapter.

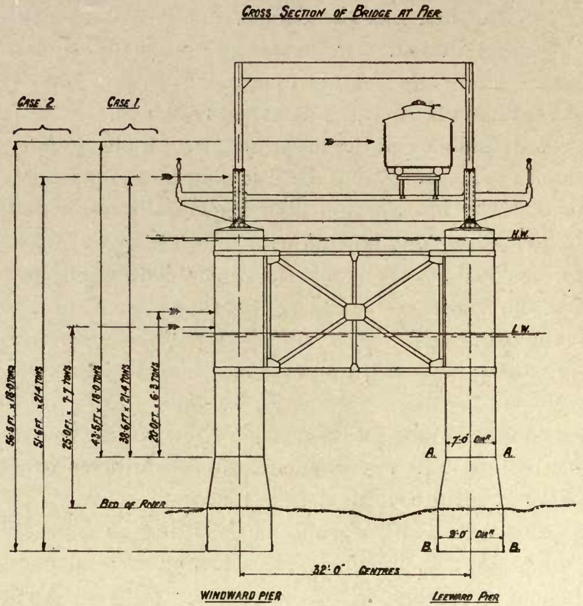


FIG. 5.
(For description, see p. 135.)

The strength and other properties of materials in ordinary use for foundation structures are given in Table III.

CHAPTER VI.

PILED FOUNDATIONS.

PILES driven into the ground are used extensively in bridge foundations, for both permanent and temporary works. They are made of various materials, and are used for different purposes. They may be classified as follows: (1) Bearing piles; (2) Reinforcing piles; and (3) Sheet or sheeting piles. Reinforcing piles need only be mentioned. They are used partly to consolidate loose ground, and partly to carry load. Their length is not usually very great, and they are driven at intervals, working from the edges of the foundation inwards. Reinforcing piles are of doubtful value. Sheet piles are for sustaining lateral pressure and will be considered in the next chapter. Bearing piles and their application will be discussed here.

Bearing piles, as their name implies, are for carrying loads. They are of two kinds—piles driven through a softer material to a hard bed, and piles driven into a great depth of soft material or of clay. The former obtain their principal resistance from the hard bed, the softer surrounding material preventing flexure. The latter depend for their value on skin friction. With piles of this kind driven into clay overlying a soft wet stratum it may be undesirable to pierce through the clay.

The bearing power of piles may be ascertained by experimental loading of representative piles; or it may be estimated by formulæ from data observed during driving. These formulæ are based on the work done in driving, and connect the penetration of the pile with the force of the blow. The factors involved are the weights of hammer and pile, the distances through which they move, the elasticity of pile and hammer, and the inertia of the pile. Some of these factors are uncertain, and the various formulæ devised differ from each other in the total disregard or in the value ascribed to certain of them. The formulæ give divergent results. Perhaps Rankine's is as correct practically as any other. Table IV gives the more usual formulæ. However divergent these results are the penetration under a measured blow is an indication of bearing capacity, and this is frequently specified. The following are some specifications:—

Waterford Bridge: total set for last 10 blows of 2½ ton hammer, 4 feet drop, not to exceed ½ inch.

Breydon Viaduct: total set for last 6 blows of 1½ ton hammer, 5 feet drop, not to exceed 1½ inches.

Another is total set for last 10 blows not to exceed 1½ inches, hammer 25 cwt., drop 7 feet: for load on pile of 30 tons.

Swing Bridge over River Avon, Bristol: total set for last 5 blows of 1 ton hammer, 10 feet drop, not to exceed 1 inch, maximum load on pile 25 tons.

Driving tests should be taken after the pile has been for some time at rest. In order that a fair result may be obtained, the head of the pile should be hard and sound, not bruised by previous driving;

and the fall of the hammer should not be impeded by friction on the guides. The penetration also should be measured several feet from the top of the pile. Rankine gives the limit of safe load for timber piles driven to a hard bed at 1000 lb. per square inch of sectional area, and 200 lb. for piles depending only on surface friction. Driving to refusal, or to a very small set, is not to be recommended, because the pile may be weakened by shattering or by the brooming of the point.

Bearing piles are most frequently made of timber, because it has the necessary strength, is easily prepared and handled, and is comparatively cheap. Metal and reinforced concrete piles are also used. The kind of timber employed is a matter of locality. Sawn pitch pine, Oregon pine, karri wood, and jarrah wood, are used in this country; teak in India and Burma; white oak in America; and ironbark and turpentine wood in Australia. Natural sticks, often of larch, barked or unbarked, are used as well as sawn timbers. Jarrah and karri are Australian timbers of the genus Eucalyptus. They have greater strength than Oregon or pitch pine, and resist the depredations of wood-boring shrimps found in many of our tidal waters. The last may be a serious matter with piles which are not driven their entire length into the ground. Wastage of pitch pine piles to the extent of 10 per cent in twenty months from this cause has been noted in the Walney channel. Karri or jarrah would only be used in preference to pine where such destruction might be apprehended. Lengths obtainable often determine the use of Oregon in preference to pitch pine. The latter is becoming increasingly difficult to obtain in long lengths, say over 40 feet, whereas Oregon can be obtained up to 60 feet. Teredo and liminoria are very destructive in Australian waters and protection has been obtained by sheathing with copper or muntz metal, and by enclosing in armoured concrete. Piles may be preserved from general decay by creosoting.

Piles are shod and ringed for driving. In soft ground the shoe might be omitted, the timber being pointed. In very soft ground even pointing may not be necessary, and the bearing can be increased by means of cleats bolted to the pile some distance from the point. Driving is accomplished by drop or gravity hammers working between upright guides, or by steam or other power hammers similarly applied. The ordinary gravity hammer after being hoisted the required height above the pile is released from the hoisting rope by a trigger and falls on the pile; in the Sykes' and other types, the winch drum is thrown out of gear and the rope is paid out with the hammer; while the Sisson's hammer has a continuous moving chain with prongs which pick up and drop the hammer at regulated distances. About 30 blows per minute can be given with the Sykes' hammer, while the trigger hammer is much slower. There is no general rule for the weight of hammer to be used. Hammers range from about 15 cwts. to 5 tons, the lighter ones being used for small piles, and the heaviest for large reinforced concrete piles. A hammer of about 2½ tons weight should be used for 12 inch or 14 inch piles of 30 feet length and upwards. A heavy hammer with a short drop, say of 3 feet, is much less likely to shatter a pile than a lighter hammer with a longer drop.

Bearing piles can be used in many cases to form or help to form a secure and economical foundation. The total load must not be too great because only a limited number of piles can be driven in a given area. For example, suppose 15 tons is reckoned as the safe load per pile, and 3 feet centres the closest pitching; then the average load per square foot would amount to only 1·66 tons. Another condition is that the piles must be supported laterally.

The following are some of the ways in which piles are used: (1) Driven at intervals into the bed under a solid foundation structure; (2) a small number driven close together and brought up to support the superstructure, the part above ground level being enveloped in a cylinder for lateral strength; and (3) driven in groups, brought up to water surface, and braced together under water.

(1) This is a very common type of abutment foundation, and is common also in America for pier foundations, although not now much used for piers in this country. Generally, loads on abutments are less than pier loads, and there is no danger to be apprehended from scour. The piles may be upwards of 60 feet in length, placed at centres of about 4 feet. The heads of the piles may be bound to each other by horizontal timbers, or may project into the concrete base. The pier foundations of Breydon Viaduct built in 1902 by Mr. Alexander Ross, M.Inst.C.E., are piled in this way. A description, giving the reason for the choice of the type, will be found in the Proceedings of the Institute of Civil Engineers,

Vol. CLVII. The piles are 12 in. × 12 in. p.p., about 40 feet long, and were driven through mud to a bottom of grey sand and stones, from the inside of coffer-dams at a level about 20 feet below high water. The load allowed per pile was proportioned to the bearing capacity, and varied from 6·81 to 12·21 tons. An American example is found in the Walnut Street Bridge over Fox River, City of Green Bay, Wisconsin. It is illustrated by Fig. 19, pp. 70 and 71. Stability is obtained in this case from the rip-rap thrown round about the piles. The piles are cut off level under water by means of a circular saw revolving on a vertical shaft, rigged out from a floating craft. Piles are used, also, under timber caissons in heavy and important foundations.

(2) A good example of the combined pile and cylinder type will be found in Waterford Bridge, built in 1911, from designs by Messrs. L. G. Mouchel & Partners. Particulars will be found in Engineering, Vol. XCI, p. 505. Each ordinary pier has four sets of three piles grouped together, the enclosing cylinder being 5 feet 6 inches outside diameter. Each special pier for the opening span is composed of four groups each of seven reinforced concrete piles, 16 in. × 16 in. × about 64 feet long, driven to rock about 40 feet below high water o.s.t., through a depth of about 30 feet of muddy sand and other materials. The enclosing cylinders, also of reinforced concrete, are 7 ft. outside diameter × 4 in. thick, sunk a few feet into the river bed, the inside spaces being filled with concrete. Fig. 6 is a horizontal section.

(3) This type amounts to a skeleton pier between ground and water surfaces, and would be used for small spans in shallow water, usually of timber where timber was abundant and other materials expensive. Efficient bracing under water is practically impossible, and this consideration would limit the adoption of the type to waters of no great minimum depth.

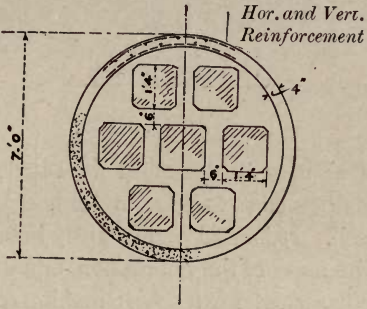


FIG. 6.—Section of pier for opening span, Waterford Bridge.

CHAPTER VII.

DAMS.

Sheet piling may be described as a series of piles driven together edge to edge to form a continuous sheet or wall. It is used permanently in bridge work to retain the mass of the foundation, and temporarily —perhaps to a greater extent—in making coffer-dams. In both cases the loads or pressures to be sustained are lateral in direction; and while in the former case water-tightness may not be necessary, in coffer-dams it is essential. It is necessary only to consider sheet piling as applied to dams, and this will be done in the course of the present chapter.

A dam for a bridge foundation is a temporary structure built to exclude water from the foundation site which it encloses. The design depends mainly on the head of water to be excluded, and the limiting condition in this direction may be taken at about 40 feet, although much deeper dams have been constructed. The design will be influenced more or less by the character of the water, and by the materials to be used in its construction. Timber and earth are the commonest materials. A varying tide of appreciable range will cause varying stresses with consequent movements of the structure; and these must be prevented in timber coffer-dams by ties of chains or rods taken from side to side to keep the whole tight. An earthen dam would be unsuitable in rough water or in a fast current. In the first place, a dam has to be emptied of water by pumping, and this is minimized in a tidal dam by having a sluice at low-water level, so that it is necessary to pump below this level only. A sluice is necessary also in emergency for flooding a dam at any time should it show signs of failure. Leakages through the sides, and seepage and springs from below have to be dealt with constantly. In order that pumping may not disturb the foundation under construction, accumulating water is led in open drains or through pipes to a sump or pump-suction pit in a convenient position inside the dam, the bottom of the sump being underneath the bed. Dams may be whole-tide—designed for continuous working ; or half-tide—designed only to exclude water up to about half-tide level. Half-tide dams are flooded above the determined level, and after the tide goes back the dam is pumped dry. A half-tide dam may be economical where there is a great range of tide with shallow water at low tide and intermittent working is admissible, because the saving in first cost may be considerable and the pumping at low water comparatively insignificant.

In still water for heads not exceeding about 10 feet, a clay, or even an earth dam with an impervious dressing on the outer face, may be used. The earth or clay is cast on each face to its natural slope, so that in confined situations such a type would be unsuitable. Where the head is inconsiderable, the excavated material banked round the site may be sufficient, with the aid of a hand pump to throw out the leakage.

Dams of timber frames with double sheathing let down and held down on a bedding of clay, or concrete, the space between the sheathings being filled with puddled clay, are sometimes made. This type has been used in the construction of some of the Paris bridges for heads up to 20 feet. It requires a levelled site, or the lower edges shaped to the contour of the bottom, and is suitable for a rocky site where piles cannot be driven.

The usual type of dam is the sheet-piled coffer-dam of half timbers or whole timbers in single row, or of two or more rows with clay filling between. Single row half-timber dams have been used successfully with a 30 feet head, as at Walney Bridge, and single row whole-timbers for heads exceeding 40 feet, as at Vauxhall Bridge. Multiple row dams were used at London Bridge. Strength and stability of a timber dam are secured partly from frames of horizontal struts placed at intervals in its height, these being closer towards the bottom where the pressures are greatest, and partly from the sheeting itself.

Sheet piles of rolled steel sections interlocked are coming into general use for coffer-dams, and their use will be extended as contractors become familiar with their advantages, largely economical, and as makers improve their construction in detail. Steel piles may be used for conditions too severe for timber piles, or where the available space is restricted. The principal advantages of steel over timber piles are these: (1) Ease of driving, because much less ground is displaced and friction is less; (2) Economy, because they can be drawn and redriven very many times, and when useless for redriving have a salvage value equal to about 33 per cent of the original cost. Timberpiles may be used twice, but their salvage value after use is slight, andthe first cost of suitable timber is increasing every year. In addition to the advantages already mentioned, obstructions are more readily cut through by steel piles, or if diverted by an obstruction are more flexible than timber in maintaining the continuity of the dam. Cross strutting is needed at only wide intervals, but fixtures are not so easily made with steel as with timber. Sections of various steel piles are given below.

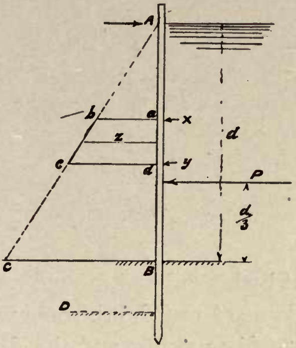


Fig. 7.

Close calculations cannot be made for the strength of a dam because so much depends on loads which can only be estimated approximately, and also on the actual treatment to which any dam may be subjected. The side of a dam is shown in Fig. 7 where d = depth of water, w = weight of a cubic foot of water, generally 64 lb., and unit length = b or 1 foot. The pressure = P, due to the hydrostatic head, is obtained from the following equation :—

$$P = \frac{wd^2b}{2}$$

(or since b is unity)

$$= \frac{wd^2}{2}. \quad . \quad . \quad (8)$$

It will increase from zero at water level to a maximum at the bottom, and is represented by the triangle ABC. The centre of pressure will be $\frac{d}{3}$ from the bottom, and if the side is not supported above, the reaction (R_1) at B = P. Apply a horizontal strut at A, and the reaction (R_2) there will be :—

$$R_2 = \frac{wd^2}{2} \times \tfrac{1}{3}$$

$$= \frac{wd^2}{6} \quad . \quad . \quad . \quad (9)$$

and

$$R_1 = \frac{wd^2}{2} \times \tfrac{2}{3}$$

$$= \frac{wd^2}{3} \quad . \quad . \quad . \quad (10)$$

assuming that both strut and ground are unyielding.

In actual dams of any depth there will be several tiers of intermediate struts, and the diagram Fig. 8 will give points of equal pressure approximately. This diagram will serve to indicate where wales and struts should be placed.

Excavation is carried down inside the coffer-dam below the ground surface to a greater or less extent, as at D (Fig. 7), and the action of the ground outside is that of a surcharged embankment. It is not calculable, however, because the nature of the soil cannot be known, but the resulting pressure with semi-liquid material may reach twice the water pressure. If this material overlies a hard bottom, like rock, down to which the excavation is carried, the piles may be forced inwards gradually before strutting can be placed.

The cross struts bear against longitudinal walings which distribute the load over the piling, and the result of the pressures is to create bending and shearing in the piles and walings, and direct compression in the struts. These cannot be calculated with any exactness, because they depend largely on the degree of tightness to which the struts are wedged up. Assume that the struts are unyielding and would carry their intended share of the loads, and disregard the deflection of the walings. The piling is then in effect a continuous girder, and between any two walings the bending moment M will be approximately

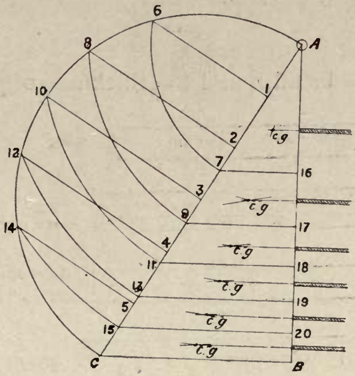


FIG. 8.

$$M = \tfrac{3}{4} \cdot \frac{Pl_2}{8}. \qquad . \qquad . \quad (11)$$

where P = average pressure of water over the bay, which can be found graphically from Fig. 8, and l = depth of bay. Over the walings M would be the same.

On timber piling the fibre stress f can be found from :—

$$f = \frac{M}{Z} \quad . \quad . \quad . \quad (12)$$

where

$$Z = \frac{I}{y}$$

$$= \frac{bt^2}{6} \quad . \quad . \quad . \quad (13)$$

where b = breadth and t = thickness of piles.

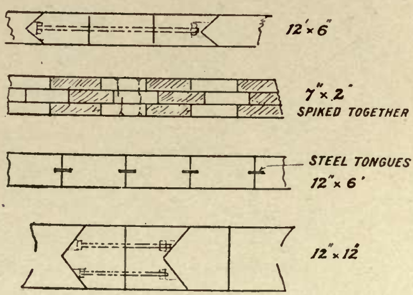


FIG. 9.—Sections of timber sheet piling.

In practice the walings and struts would be placed at distances calculated to limit the greatest fibre stress on, and also the deflection of, the piles decided upon : allowances being made for slack struts, etc. f in tension (f_t) for p. p. timber is about, say, 8 cwts. per square inch safe stress.

Timber piles are driven in combinations of two or three, bolted together at intervals which depend on the diameter of the bolts and the stiffness of the piles ; $\frac{7}{8}$ inch bolts at 4 feet centres may be used for 6 inch p. p. piles, heads and points being sunk flush. The intermediate joints of piles are planed fair, and heavily coated with red lead, or other insertion. The driving joints may be dressed plane and afterwards caulked or may be veed. Fig. 9 shows usual sections of timber sheet piling. The point is shaped and shod at an angle, as shown in Fig. 10, to press the pile being driven against its predecessor, and the spigot is driven against the faucet part of the joint; king or guide piles are first driven at distances of about 10 feet. These king piles are stronger than the others, their function being to keep the piling from getting out of line and to provide fixing for the temporary longitudinals needed to guide the intermediate piles. The last pile of a bay or division is made to fit the space between the king pile and the second last pile, and in addition is given a taper in its length to wedge all the joints tight.

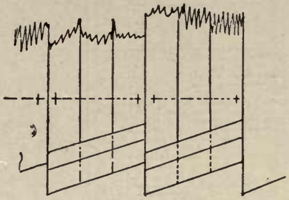
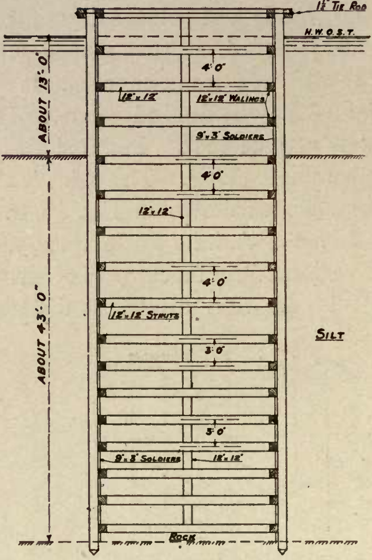
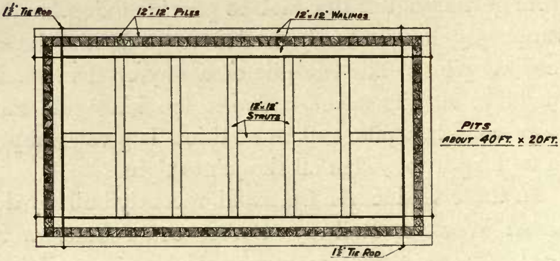


Fig. 10.—Points of timber sheet piles.

In three viaducts in Cornwall, recently built by the Great Western Railway, timber coffer-dams were used. That at St. Germans was the heaviest, and had eleven piers, four being deep. Fig. 11 shows the coffer-dam used. The contractor used about 84,000 cubic feet of timber. The foundations for the piers of the via-



SECTION



PLAN

FIG. 11.—Coffer-dam used at St. Germans Viaduct, Great Western Railway.

ducts of the same railway over the Thames at Maidenhead, Goring, and Moulsford were placed inside of coffer-dams of two rows of 6 inch sheet piles, about 3 feet 6 inches to 4 feet apart, with clay puddle between, and having 14 in. × 14 in. king piles. The bottoms were in compact chalk, about 5 feet below river bed, water about 10 feet deep. Sheet piles were driven 15 feet into the bed, and king piles 20 feet. In a viaduct over the Taff at Cardiff (also G.W.R.), the piers were built inside a coffer-dam consisting of a single row of whole piles, with meeting sides planed to fit close, the water swelling the timbers to a tight fit. These foundations were in coarse gravel. The sites were clear except for about 18 inches of land water at low tide, but at high water the tide would be 10 to 12 feet. Six river piers were taken about 18 feet below the bed, so that the total head was about 30 feet. The trenches were 45 ft. × 9 ft., and 12 in. × 6 in. waling with 12 in. × 12 in. struts were used.

Sections of a number of patent steel sheet piles on the market are shown in Fig. 12, and Table V gives some of their properties compiled from the trade publications of the makers. Special corner and junction pieces are made up from the various sections. The requirements of steel piling are that it shall have sufficient lateral strength when driven, that it shall be stiff enough to stand driving without inconvenient steadying, and that it shall be practically watertight. In the last connexion, the clutch or socket must be free enough for driving and at the same time form a close joint. It should, also, be very substantial in order to develop the strength of

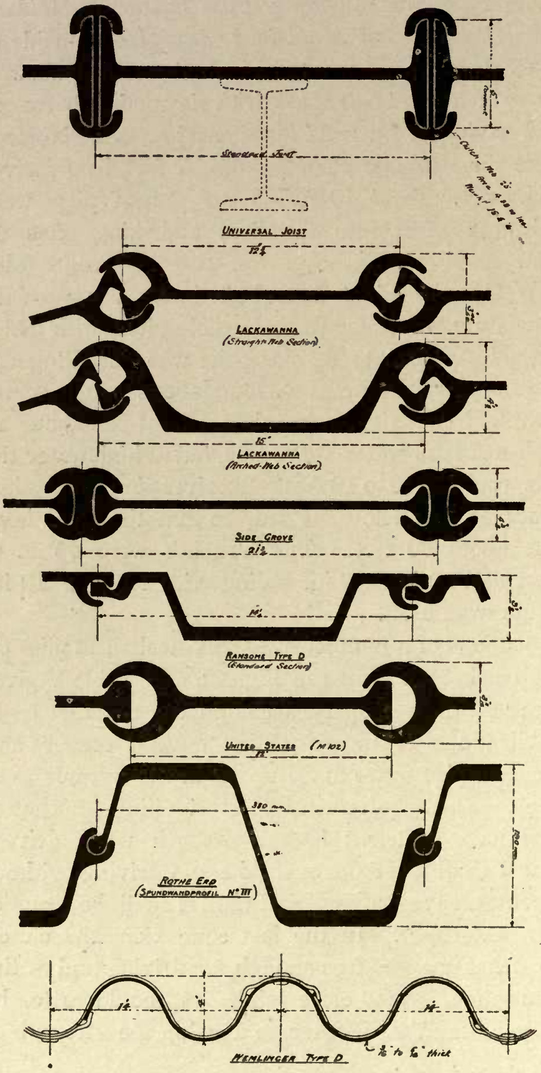


Fig. 12.—Sections of rolled steel sheet piling.

the piling. Slight irregularities in rolling cannot be altogether avoided, but the grease used to facilitate driving helps to make a joint, and slight leakage can be closed up by means of ashes or other substances thrown into the water outside, as is done with timber dams.

In the "Universal Joist" piling, of the British Steel Piling Company, the clutches are all alike, weighing 15½ lb. per foot run, but ordinary joists of any depth with a 5-inch flange may be used. The "Universal Compound" as indicated by the dotted lines in the figure is for heavy work. The former type of piling was used for the abutment and pier foundations of the Deptford Creek Bridge, L.C.C. The dam for the pier was 80 ft. × 40 ft. inside, and the piles were 40 feet long driven 20 feet to 25 feet through Thames ballast into London clay. When driving was completed, and as soon as a small head of water was obtained, a few loads of ashes were cast into the creek. These were sucked into the interlocks and the dam was almost perfectly watertight, only a 4-inch centrifugal pump working intermittently being required to keep the bottom dry. Piling 15 in. × 5 in. of the same kind was used for the abutment foundations of the new bridge over the Kelvin in Kelvingrove Park, Glasgow.

Fig. 13 shows the application of Lackawanna piling in obtaining the foundation of the eight piers and the two abutments of the bridge with Strauss opening span, carrying the Lake Shore & Michigan Southern Railway over Buffalo River at Buffalo, N.Y. Each pier foundation consists of a pair of concrete cylinders, 13 feet diameter, bedded on rock about 40 feet below water level. Circular dams, 18

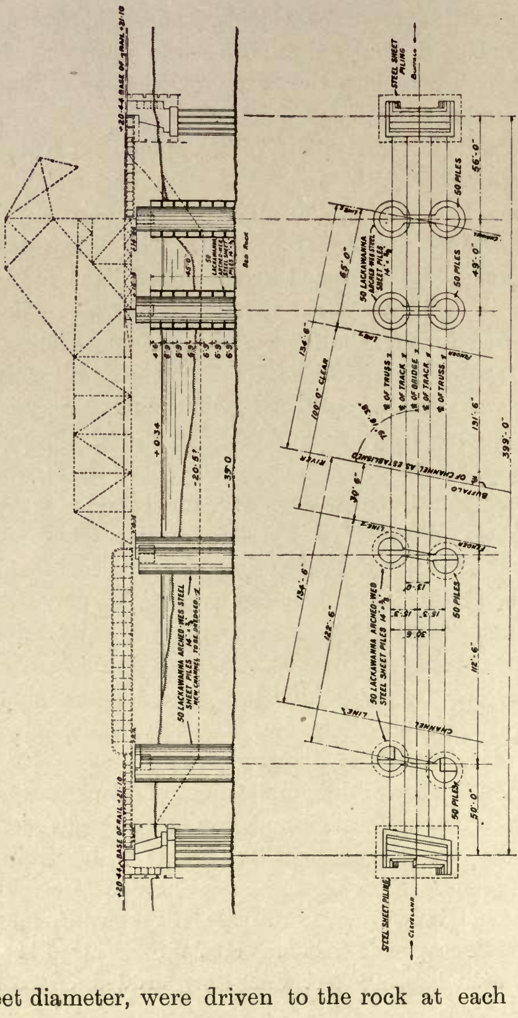


FIG. 13.—Bridge carrying the Lake Shore & Michigan Southern Railway over Buffalo River at Buffalo.

feet diameter, were driven to the rock at each site.

Timber frames, with a 10 feet square opening in the centre, were placed at vertical distances of about 5 feet. An orange-peel bucket dredger was used for excavating under water until hard pan was reached about 5 feet above the rock. The cylinder was then pumped out and excavation completed in the dry. Several other installations of this piling could be mentioned, but the illustration, Fig. 14, will be sufficient. It shows arched-web piling applied in forming the foundation of an opening bridge at 92nd Street, Chicago, Ill., where it was a most successful construction. The licencees in this country for Lackawanna piling are the Cargo Fleet Iron & Steel Company, Ltd., Middlesbrough.

The Side Groove Piling, of the Side Groove Steel Piling Supply Company, Ltd.—Annison's Patents, was selected by the sub-contractors after careful consideration for constructing the coffer-dams required for the removal of the old piers of Southwark Bridge, under reconstruction at the present time. This piling is composed of side groove pile bars with oval bulb steel sheeting driven between. The bars provide the strength, and the sheeting need not be driven as far into the bed as the bars, and the bars themselves may be driven deeper at intervals as king piles. It is, therefore, possible to effect saving in the amount of piling used as compared with other types where the complete piling must be driven right down; and for the same reason and because of its width there is economy in driving because fewer drives per lineal foot of dam are required. The clutches of this type are very strong, both at root and point, to resist deformation or opening out. The loads per foot required to pull bulb through the clutch is about 72-75

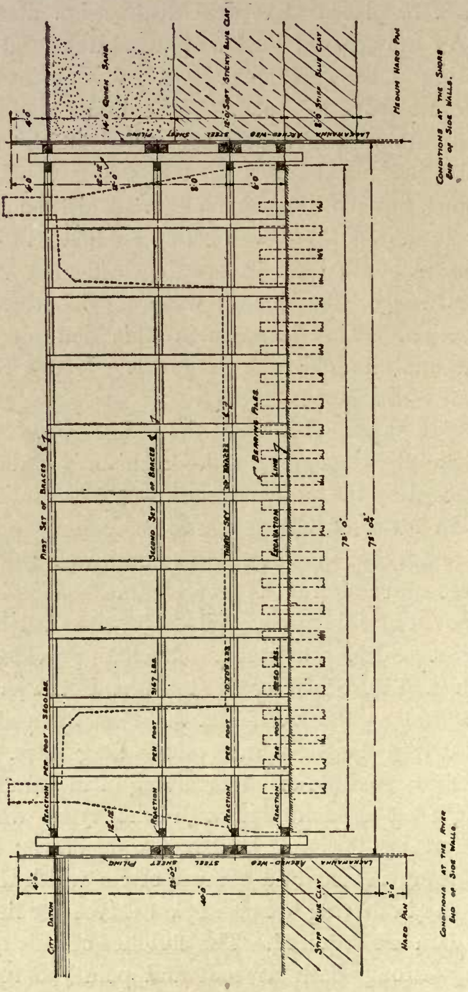

14'0" Quick Sand
12'0" Soft Sticky Blue Clay
8'6" Stiff Blue Clay
Medium Hard Pan
Conditions at the Shore End of Side Walls
Lackawanna Arched-Web Steel Sheet Piling
4'0"
6'0"
8'0"
12"×12"
First Set of Braces
Second Set of Braces
Third Set of Braces
Bearing Piles
Excavation Line
Reaction per foot = 3800 lbs.
Reaction per foot 9167 lbs.
Reaction per foot 10707 lbs.
Reaction per foot 2880 lbs.
78' 0"
78' 0½"
City Datum
25'0"
40'0"
3'0"
Stiff Blue Clay
Hard Pan
Conditions at the River End of Side Walls

FIG. 14.—Pier coffer-dam for bridge at 92nd Street, Chicago, Ill.

tons. The sections throughout are of British standard material and workmanship, being rolled at the Earl of Dudley's Round Oak Works.

The figure (12) shows the lighter section of Ransome piling. The abutment coffer-dams for Warrington New Bridge were constructed of this piling, and it was used in the widening of the Kingston-on-Thames Bridge. The piles at Warrington were 27 feet long, driven into average strata, with a tidal head of 18 feet. The proprietors are the Ransome-ver Mehr Machinery Company Ltd.

There are four weights of United States Steel Piling rolled, that figured being the heaviest. It has been used in the construction of the Sara Bridge, Lower Ganges, and of many American, Canadian, French, and other bridges abroad. It is manufactured by the Carnegie Steel Company, of Pittsburg, and sold by the United States Steel Products Company, New York.

The Rothe Erde Steel piling of the Gelsenkirchener Bergwerks-Aktien-Gesellschaft, and the steel sheet-piling of the Wemlinger Steel Piling Company, New York, are also suitable for bridge foundation work.

It should be said that the sections illustrated have perhaps found their greatest field in work outside of, although similar to, bridge foundations; and a fair comparison of the extent of their use is not intended in the preceding notes. Such a comparison should be made from a study of all available information, including prices, applied to the conditions of any particular case. The handbooks of some of the firms mentioned give valuable general information.

Sheet piles are driven in much the same way as bearing piles—by means of hoist or steam operated gravity hammers, double acting hammers, and also by percussion piston hammer moving down with the pile. The clutch should be driven over the solid edge where possible—that is, the solid edge should always lead, because the clutch is in this way prevented from being choked with soil or stones. A weak interlock is apt to result in deformation and burring during driving, and this, besides creating a difficulty for redrawing, lessens the useful life of the piling. Rapid short drops are most effective in driving, and if obstructions are encountered it is much better to increase the rapidity of the blows than to lengthen the drop. Since the principal advantage of steel piling lies in the repeated use of it, the withdrawing of the piles is a serious matter. Over-driving should not be indulged in, and attention should be given to the lubrication of the driving joints. Concrete set for any depth against the piling will render withdrawing almost impossible. Cranes and tackles can be used alone, or in conjunction with levers for withdrawing piles. The Ransome-ver Mehr Company make a pile extractor which is percussive in action, and is slung over the head of the pile.

CHAPTER VIII.

SCREW PILES AND SCREW CYLINDERS.

UNDER this heading are included disc piles or cylinders in addition to screw piles and screw cylinders. A screw pile consists of a shaft which is sometimes of tubular section, but usually of solid round wrought iron or steel 3 inches to 8 inches diameter, to the lower end of which a cast-iron or cast-steel screw blade is fixed. As the name implies, the pile is forced into the ground as a nail is screwed into a piece of wood. The shaft must be strong and stiff enough as a column to support the load, and also to withstand the torsion caused by screwing. The blades must have an area determined by the load to be carried and the bearing value of the ground. The pitch of the screw and the general form of the blade and point should be designed to suit the ground. The blade should not have more than one complete turn, otherwise it is liable to be broken by stones getting into the screw. The edge of the blade may be serrated to assist screwing into hard ground. A small pitch screw will be more easily turned than one of a large pitch, because the work done per turn of the shaft varies directly as the pitch of the screw or the distance travelled. The full pitch will not be gained for each turn. The slip will be greater or less according to circumstances.

A disc pile or cylinder is usually a hollow shaft or column of from about 10 inches to 15 inches diameter with a disc or flange 2 feet 6 inches to 3 feet 6 inches diameter at its lower end. There are one or more orifices through the flange inside the shaft, and the lower face of the disc may be serrated or have projecting teeth.

The screw cylinder in effect is a larger pile. It consists of a hollow cast-iron cylinder, say from 1 foot 2 inches to 2 feet 9 inches diameter, with open lower end and a screw blade of 4 to 5 feet in diameter, a short distance from the bottom edge.

Screw and disc piles and cylinders are used for small span bridges, where the total loads are not very great, in suitable beds. The most suitable bed is silty sand, or other similar homogeneous material, where no obstructions such as stones or boulders are to be met. Their use, however, has not been limited to such materials because instances could be given of piles put down in clay formations with boulders, chalk, and other difficult beds. Actually 18½ feet into chalk, the lower half hard, has been reached by screwing, but this could not be an advantageous instance of their use. Piles and cylinders are disposed in rows or groups. They offer less obstruction to a current than solid piers, and can be erected quickly. They correspond in some respects to a braced timber pile pier, but are of more durable character. In deep water, connecting bracings for stiffness can be fixed only with difficulty. As has been indicated above the bearing value of a pile, which can be ascertained by experimental loading, depends on the area of the blade and on the resistance of the ground. This is increased somewhat as a result of the screwing. About 5 tons per square foot is a reasonable allowance in most cases. The blades should be beyond the reach of scour or other disturbance.

Screwing down is carried out from a fixed or floating barge or platform at the site by various means. The usual way is to fix a capstan head or wheel of large diameter to the shaft, round which one end of a rope is wound. The other end of the rope is led to a winch conveniently situated. Two winches leading off from each side of the wheel may be used. Hoisting on the winch imparts a turning motion to the shaft and screw. The same result is attained by capstan bars placed in the capstan head and pulled round by tackles. Spur and worm gears have been used, and these impart a steadier motion to the pile than the wheel or capstan head. Hydraulic power has been successfully employed, acting on twin cylinders having rams with hinged pawls engaging in a ratchet wheel attached to the pile. It is sometimes necessary to load the pile, so that it may bite while being screwed into stiff ground. In very hard ground screw piles have been hammered down by a light piling hammer while being screwed.

Disc and screw cylinders are sunk by the combined use of water jet and turning, except where water jet is sufficient alone. The water is usually taken down the centre of the cylinder in a pipe of about 2 inches diameter, and out a short distance beyond the end; in other cases the cylinder itself is filled with water from a pipe screwed into a flange on the top. A flexible connexion is taken to the pumps to allow for sinking. The pressure and volume of water used vary greatly. The former may range from a few pounds in sand to perhaps 50 lbs. per square inch in clay. Due to the reaction of a high pressure in stiff ground, the pile of cylinder would require to be heavily loaded.

Screw piles, as made by the Barrowfield Iron Works, Ltd., Glasgow, are illustrated in Fig. 15. Fig. 16 is a screw cylinder as used in the Usk Viaduct, Great Western Railway, near Newport. Fig. 17 gives general and detail views of screw cylinder piers of the Chiromo Bridge, Shiré Highland Railway, of which Sir Douglas Fox & Partners were engineers. The bottom was sand, and capstans

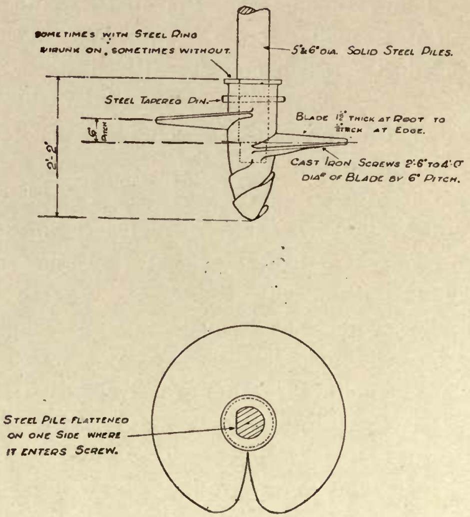

SOMETIMES WITH STEEL RING
SHRUNK ON, SOMETIMES WITHOUT.
5" & 6" DIA. SOLID STEEL PILES.
STEEL TAPERED PIN.
BLADE 1½" THICK AT ROOT TO ½" THICK AT EDGE.
6" PITCH
2'-2"
CAST IRON SCREWS 2'-6" TO 4'-0" DIAᴿ OF BLADE BY 6" PITCH.
STEEL PILE FLATTENED ON ONE SIDE WHERE IT ENTERS SCREW.

Fig. 15.—Screw pile.

were used in screwing. The loads on a rest of six

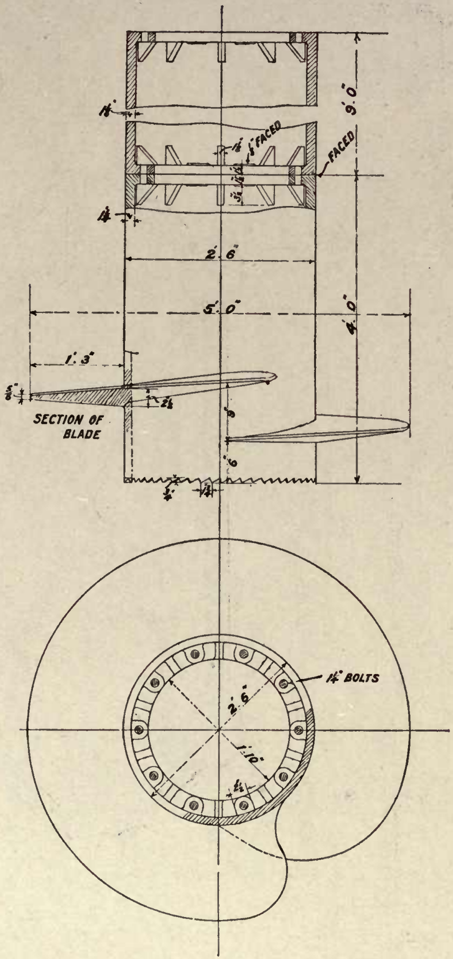


Fig. 16.—Screw cylinder.

cylinders under each end of lifting span are these:—

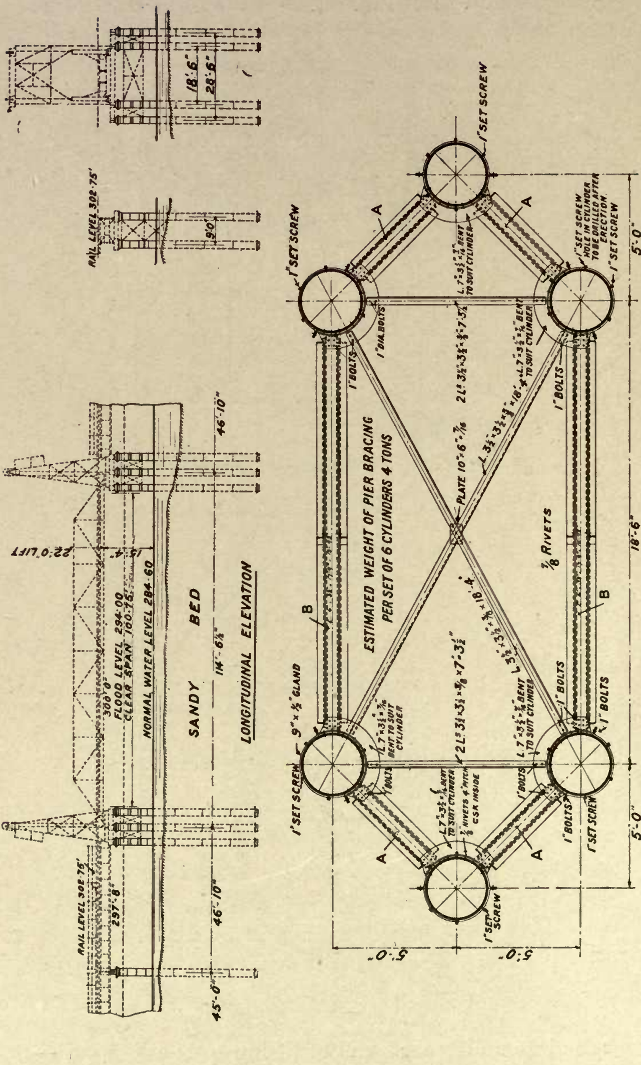


Fig. 17.—Chiromo Bridge, Shiré Highland Railway.

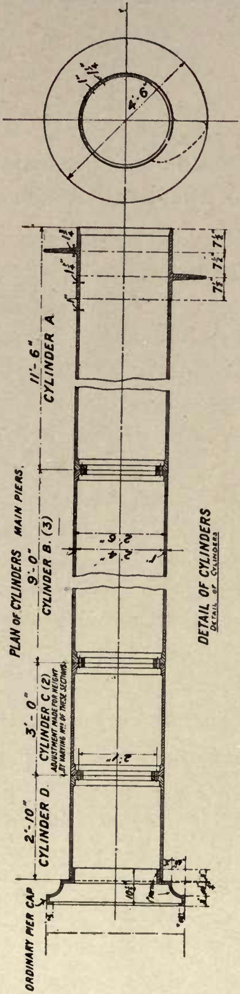


Fig. 17—*continued.*

Dead load, including cylinders .	210
Live load	126
Impact	42
Tons	378

or $\frac{378}{6}$ = 63 tons per cylinder.

On a rest of two cylinders between ordinary spans the loads are :—

Dead load, as before . . .	52
Live load	75
Impact	29
Tons	156

or $\frac{156}{2}$ = 78 tons per cylinder.

CHAPTER IX.

WELL FOUNDATIONS.

Well foundations consist of hollow cylinders of brick, stone, or concrete masonry, on a curb of timber or metal, built upon dry ground or upon an artificial bed raised above water, and sunk to intended depth by the removal of the soil inside and their own weight, assisted if necessary by temporary loading. A section of well is built on the curb, the mortar allowed to set, and the section sunk. Another section is added and sunk, and so on. After the well has reached its final depth, it is cleaned out and filled with a hearting of concrete or other material. Foundations of this kind have been made for centuries in India for river walls and other purposes; and the method is native also to Egypt, Burma and elsewhere. They are well suited to alluvium and silty soils, and have the great advantage of using local materials, which can be easily and quickly obtained, and of employing native labour skilled in the method. In India the native sinker works as far as possible with a phaora or hoe, and after a depth of about 5 feet uses a jham. After taking a deep breath, he descends with the jham, pushes it into the sand with his feet, and returns to the surface of the water while the men above pull up the full jham. The daily rate of sinking of Indian wells has been stated at from 6 inches to 2 feet 6 inches. One way of loading is to form double basket work walls round the top of the cylinder and cast the excavated material into the annular space between. The limit of usefulness of the native system is probably in the neighbourhood of 20 feet depth.

In Egypt, the hollow cylinders are of native brickwork on timber curbs. The basket for the excavation is hung from a winch straddled across the top of the well. The Arab diver goes down into the water on the rope, and either pushes the basket into the sand with his feet or grabs it full with his hands. He then goes up far enough to breathe, and the basket is pulled up and emptied. Sometimes the well is unwatered or partially unwatered by bucketing or pumping.

British engineers in India early adopted the method and extended its scope, and a large proportion of Indian bridges are founded on wells. The native diver has been superseded by modern appliances, such as grabs, excavators, and sand pumps, and the construction of the wells has been altered in detail. Wells have been taken through clay and other difficult materials, but only slowly and expensively; and it has appeared sometimes as if Indian engineers had held to the method when the pneumatic process should have been employed. As the depth increases the cutting edge has to be cleared by helmet divers, or after pumping out, so that the well with its superincumbent loading may have skin friction only to overcome. In clay, gravel, and other obdurate materials the difficulties of excavation, clearing the edge, and sinking are enormously increased. Blasting may be necessary, and the sinking load has to be increased as much as possible. The difficulty and danger of piling up the great height of loading sometimes needed, and the expense incurred, will be apparent. Partial unwatering ordinarily aids the sinking, because the outside water coming in under the curb tends to loosen and clear the material, displacement is lessened and the effect of the weight increased, and the pressure on the soil inside is decreased, rendering it easier for the grab. But there is a danger if unwatering is carried too far of serious blows occurring, which may fill up the cylinder and bury the gear, and otherwise cause delay and expense.

Experience has shown that wells should not be placed too close together, because the sinking of wells plumb is a practical impossibility, and there must be room for errors in this respect. For the same reason an ample margin in diameter, say at least 12 inches, should be allowed for all round the upper substructure, so that the latter can be started fair. Wells are said, also, to draw towards each other if too close, and 3 feet has been given as the proper minimum distance between adjacent wells. Single rows of wells, with smaller wells returned under wing walls, are adopted for abutments. Twin or triple wells may be used for piers, or only a single well of larger diameter. A single well presents an equally rounded face in every direction, which is an advantage where the current may run in any direction. The amount of masonry is less in a large well than in a number of smaller wells of an equivalent area in aggregate, and the area exposed to surface friction is less.

The depth to which a well must be sunk is determined by the depth of probable scour, and the well must be designed to withstand overturning at this depth, which will usually be found sufficient for the direct loads. Skin friction cannot generally be depended on owing to the movement of the sand. The curb or cutting edge bolts are carried up the full height of the masonry to reinforce and bind it, and annular ties are built into the masonry at intervals in the height, with the same objects.

The following are some examples of well foundations. The Empress Bridge built in 1874-78 over the Sutlej has 16 spans of 264 feet c.c. piers. The 17 abutments and piers are each founded on triple wells taken down through sand, clay, silt, and sandy silt to blue clay at depths varying from 115 to 118 feet below low water. The wells are of brickwork, 19 feet o.d., and 8 feet 3 inches i.d., on curbs of deodar timber 3 feet wide and 3 feet 6 inches deep. Twelve bolts, $1\frac{5}{8}$ inches diameter are carried from the curb to the top, with tie plates between adjacent bolts at intervals in the height. After excavation, the bottom 20 feet of the well, plus the dredged hole underneath the curb, was filled with hydraulic concrete, then fine sand filling, with a plug of concrete and masonry, 12 feet thick, at the top. Each well towards the bottom required an extraneous load of 400 to 1000 tons of rails to sink it. The foundation work occupied three seasons. Sand was dredged by Bell's excavator. Sinking through clay was very difficult. At first it was taken out by divers, but afterwards

Gatmel's excavator was designed, and also a side excavator for undercutting the curb. Excavation in sand was carried about 2 feet below the cutting edge, and the well unwatered as quickly as possible by bucketing to cause a run. In clay it was often necessary to go down 12 to 15 feet below the edge. On an average 75,000 cubic feet of loose stones were placed round each pier as a protection from scour. The average cost of each well—materials, sinking, stone protection, and everything included—was fully £92, or 41s. per cubic yard of water and soil displaced. Sinking alone, with loading, cost 13s. per cubic yard, 11/20th being for sinking, and 9/20th for loading. (See Proc. I.C.E., Vol. LXV.)

The Koyakhai Bridge, Bengal-Nagpur Railway, 1897-99, has 20 spans of 160 feet c.c. Each pier is founded on a single masonry well, 26 feet 6 inches o.d. and 13 feet 6 inches i.d. on iron curbs. Each well sunk through an average depth of about 45 feet fine sand and 34 feet clay. Thirteen are founded on the clay, and eight on material below the clay. Bruce 3-bladed dredgers were used in the sand, but made no impression on the clay. Special spears constructed of rails were made, therefore, to break up the clay preparatory to dredging. Average cost of sinking per 100 cubic feet of well displacement, including coal and consumable stores, was Rs. 12.938 or 4s. 5d. Rates of sinking in sand to clay about 1 : 3; cost of fuel in the two cases about 7 : 13. Engineer, Mr. W. T. C. Beckett, M.Inst.C.E. (See Proc. I.C.E., Vol. CXLV.)

The bridges on the Pegu-Moulmein Railway, Burma, about 1906-8, are founded on brick wells.

Spans of 100 to 150 feet have one well, each pier 22 feet o.d., and smaller spans on wells 16 feet o.d. They were sunk to depths of 40 feet below bed of river by dredging, and afterwards filled with concrete. (See "The Engineer," June 4 and 18, 1909.)

The river Esk Viaduct, North-Eastern Railway, has 13 semicircular brick arches of approximately 60 feet span. The water is shallow at low tide and, following Indian practice, brick wells were used for the foundations of the four river piers. Three piers have each three wells, 14 feet outside diameter, and the fourth two wells 20 feet o.d. Curbs are of wrought iron, and the masonry of brickwork in cement 3 feet thick. Dredged partly by Priestman grab, and partly by special grab, through about 40 feet of gravel and silt, with beds of old oak-trees, to a freestone bed. A diver was required to cut and clear the trees. Maximum load on bed $5\frac{1}{4}$ tons per square foot. (See Proc. I.C.E., Vol. LXV.)

The abutments of one of the three Rodah bridges —that at Old Cairo—of which Messrs. Sir William Arrol & Co. were the engineers, were built on brick wells. The wells under the main portions were 10 feet diameter, four in number, and were sunk over 25 feet below Low Nile. Wells 4 metres diameter brickwork 0·23 metres thick, were sunk at the recently completed Ramaleh Bridge, Cairo, to a depth of 8 metres under water; the cost being P.T. 75 (say 15s.) per metre depth. This depth is perhaps exceptional, the usual being 4 to 5 metres.

On the Central Argentine Railway piers are usually founded on brick cylinders upon steel or reinforced concrete curbs. A common design for a double line bridge is shown by Fig. 18. Sinking is

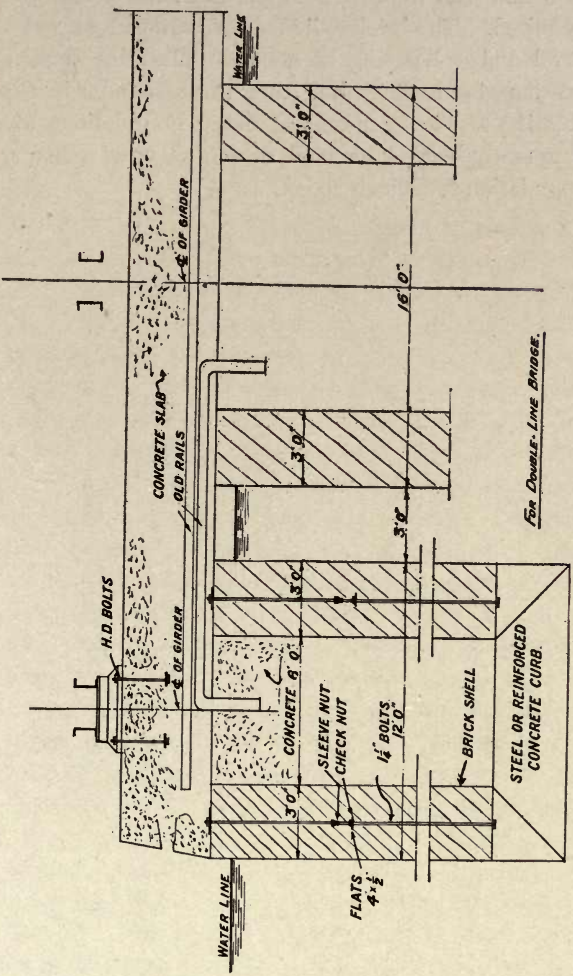


FIG. 18.—Brick cylinder bridge piers, Central Argentine Railway.

carried out by dredging and loading of the perimeter of the brickwork with rails stacked up. Between 300 and 400 tons of rails per well are sometimes required. The brick cylinders are stopped at water level and a hearting of concrete filled in—first, a bottom plug under water, then the remainder in the dry if possible by pumping out. Foundations are frequently bedded on sand or soft clay, as a firmer foundation is difficult to get.

CHAPTER X.

STRANDED CAISSONS.

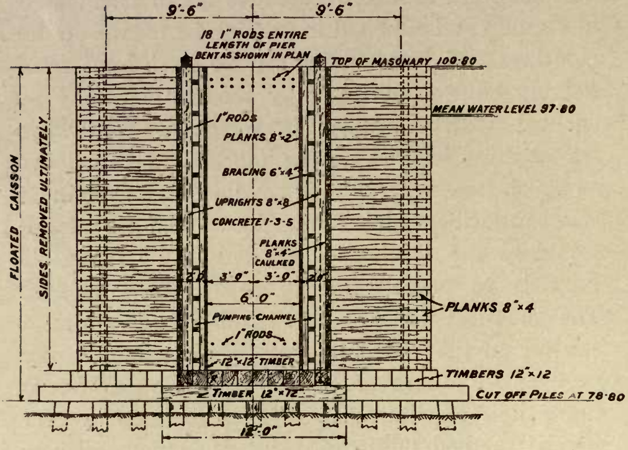
CAISSONS may be defined as enclosing shells, the main, and usually the only, purpose of which is to serve as a means for getting a foundation mass to its bed. Caissons may be circular, rectangular, square, rectangular with rounded ends, oval or elliptical, octagonal or any convenient form in horizontal section; and may have vertical sides, sides slightly inclined inwards, or bases spread out. The simpler forms would be chosen by engineers who had both to design and to manufacture the caissons. Caissons have been constructed of cast iron, wrought iron, mild steel, timber, and reinforced concrete. They may be classified according to the method by which they are sunk as follows: (1) Stranded caissons; (2) Open caissons; (3) Pneumatic caissons. Broadly, the first kind have a bottom, and the last a top, while open caissons have neither top nor bottom.

(1) A stranded caisson is one which is floated over its site and gradually lowered into place on a prepared bed by the building up of the foundation structure within it. The usual form is a rectangular water-tight box, with open top, made of steel or timber. The method dates back to the first Westminster Bridge in 1738, if not earlier, and was frequently used by the first generation of modern

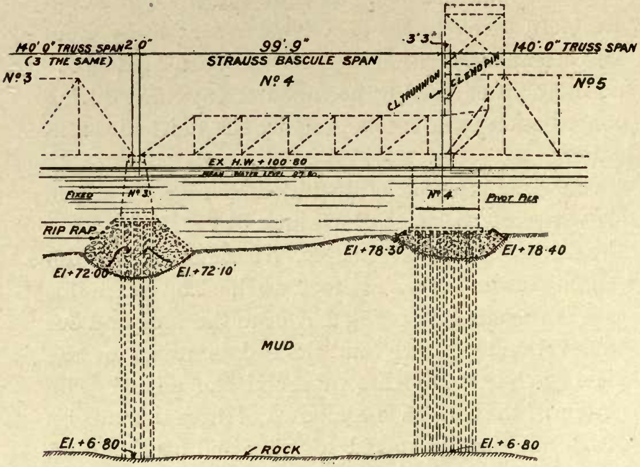
engineers. Its origin may be traced to the old barge, filled with stones, and scuttled and stranded on a levelled bed. Stranded caissons are now rarely used in this country, but are quite common in the United States, and are adopted to some extent on the Continent. The bed may be prepared by dredging away the soft top and levelling, bearing piles may be driven over the area, concrete may be deposited under water kept in position by frames or sheet piles, or bearing piles and concrete may be used together. The loads on the sides of the caisson are due to the head of water and are the same as for a coffer-dam. When the caisson is floating there will be a uniform upward buoyant pressure on the bottom, which increases directly with its depth under water. The downward loads, if the building inside is carried up solid, will be uniformly distributed all over the area and there would be no stress of any kind on the bottom. Usually, however, in a large caisson the building will be carried up all round the sides, with cross stiffening walls between. These would act as concentrated loads, and would produce bending and shearing stresses. It is unlikely, however, that a caisson would be designed from considerations like these. The sides and bottom would be made water-tight—by close riveting and caulking in a steel caisson—and the bottom be made strong enough to take up probable stresses produced by launching or lifting it into the water. Once in the water the bottom and walls, if the construction was carried up hollow, would be made of ample thickness to take any possible stresses.

This type of foundation in still water requires no false-work around it, because it can be moored like a ship and adjusted in position by means of the moorings, and the building can be carried out from floating craft. In this way the method may be adopted with advantage in restricted waterways where there is heavy shipping, in order to leave as much free waterway as possible. The depth of water in which such a foundation can be placed would be the depth at which it would be possible properly to prepare the bed—say about 20 feet. The practical difficulties of landing and setting a caisson on the bed limits its application to any but very small ranges of tide. Rough water would cause the same difficulties. It is therefore a very special method limited to still and comparatively shallow waters, where the bed can be treated in any of the ways mentioned. It admits of the foundation structure being made to a large extent away from its permanent position, and as has been said above no more space need be occupied at any time during construction than is taken up by the finished foundation.

American practice is illustrated by Figs. 19, which show two of the piers of Walnut Street Bridge over Fox River, City of Green Bay, Wisconsin, and in detail one of the timber caissons used. The bed in this case is prepared by piling driven to the rock and cut off level under water, and the bed is raised at the piers by the rip-rap thrown round the piles. The bottom of the caisson is composed of three thicknesses of 12-inch timbers spiked together and caulked, and the sides are framed with 4-inch caulked planking outside and 2-inch planking inside. The bottom



SECTION AA



ELEVATION

FIG. 19.—Piers of Walnut Street Bridge, City of Green Bay, Wisconsin.

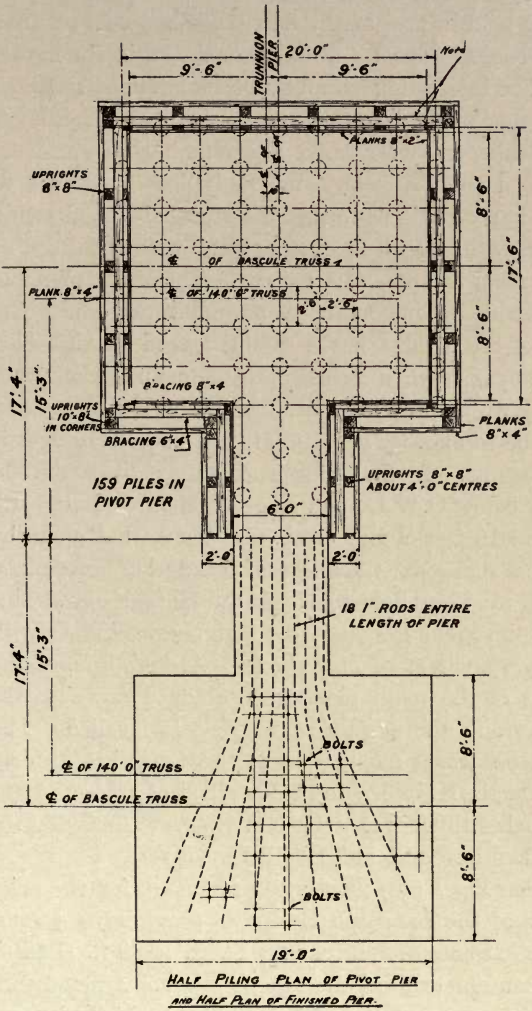


FIG. 19—*continued.*

forms a permanent grillage; and the sides, temporarily held in position by long bolts, are cast off after the concrete is set. Details are shown in the figure.

Two interesting examples of foundations obtained by this method have been carried out recently at Copenhagen by Mr. H. C. V. Mollër in the reconstruction of the Langebro and Knipplesbro Bridges in the harbour. The former has an equal-armed swing span with two clear openings of 72 feet, flanked at each end by three small spans. The centre pier was founded by means of a stranded caisson. The Langebro Bridge has a double-leaved Strauss bascule span, with a fixed approach span at each end. The two main piers were founded in a similar way to the former. The ultimate bed at both bridges is limestone. At the Langebro Bridge it lies about 38 feet below O.W.L., and is overlain immediately by about 10 feet of fine sand, and above that again by about the same thickness of hard clay intermixed with sand and limestone. At the Knipplesbro Bridge the limestone lies about 28 feet below O.W.L. with only a few feet of clay above. At both bridges the sites of the main piers were dredged down to the limestone, the surface of which was levelled. The caisson was sunk to this at the Langebro Bridge, but at the Knipplesbro Bridge a bed of concrete 4 feet 6 inches thick was deposited under water within a sunken iron crib, and levelled by divers.

The Figs. (20) illustrate the caisson for the pivot pier of the Langebro Bridge in progressive stages. The extreme dimensions are 62 ft. × 30 ft. (Danish measurement), and the caisson was constructed with a stiff bottom of H and L sections to which a bottom

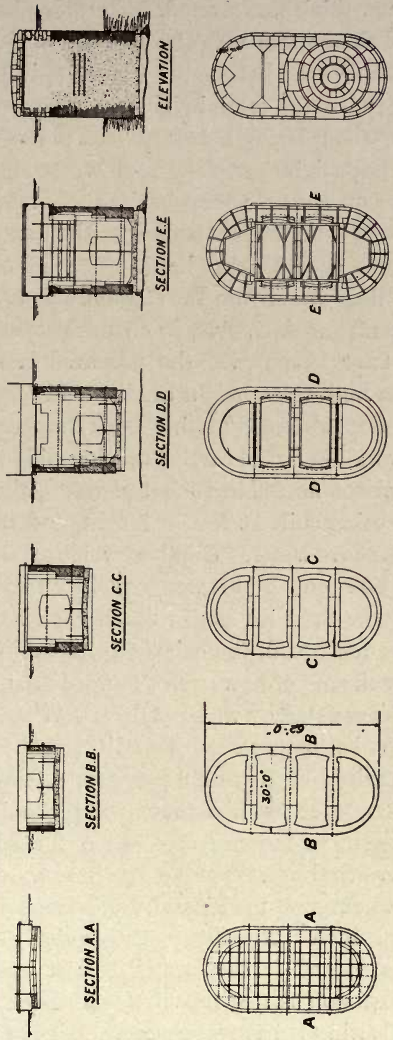


FIG. 20.—Pier for Langebro Bridge, Copenhagen Harbour.

plate 10 mm. thick was riveted. The side plate running round the bottom was also 10 mm. thick. The side plates above the bottom are 6 mm. (¼ in.) thick, carried up until the total height of the permanent caisson was about 10·25 metres. These plates merely formed a watertight envelope and were dependent for support on the masonry behind. A temporary caisson about 3 m. high attached to the top carried the structure above water level. The bottom was tested by filling water into the caisson. It has a fall from the centre to each side, to ensure proper grouting, at a later stage, over the prepared bed. The caisson was built for a height of about 11 feet at the slip of the Copenhagen Floating Dock Company, and the brickwork afterward carried up alongside the slip until the caisson floated in 16 feet of water, the brickwork then being built 13 feet in height, and the shell about 6 feet above water. The total weight at this time was about 800 tons, and it was towed by two tugs in this condition over its site in the harbour. Building up was continued, the caisson being adjusted in position by means of hawsers. Towards the bottom sinking was assisted by water ballast. When within 1 foot of the bottom an accurate adjustment for position was made, and the sinking afterwards continued. Four divers, each with a fourth of the caisson to watch, were in attendance, and when the edge was 1 inch above the highest point of the limestone a final fine adjustment was made; and the caisson lowered another half inch. The divers then quickly wedged up the caisson and further water ballast was added until the caisson rested heavily, and there was no risk of it floating if the water rose. Bags of cement were then placed around the edge by the divers to close up the spaces, and these were backed by clay and sand to a depth of 12 feet. While sinking was in progress the limestone bed had been thoroughly cleaned, and it remained now to cement grout the space between the limestone and the bottom of the caisson. The grouting was done in such a way by means of filling and outlet pipes as to expel any mud and leave pure grout. Grouting took twenty-four hours, and 930 barrels of cement were used.

The bottom part of one of the caissons of the Knipplesbro Bridge was built and launched from a slipway, while the other was built on ground adjoining the harbour, and lifted into the water by means of two floating shears.

The piers for the Bridges of Saumur, Bordeaux, and Ivry, and of Pont Jena at Paris were founded upon stranded caissons.

CHAPTER XI.

OPEN CAISSONS.

Open caissons are open hollow structures, the excavation of which is usually carried out by means of grab dredgers, but sometimes in special conditions by open excavation, or with the aid of sand or similar pumps. When of cylindrical form they are comparable with the wells already described, but are of stronger construction, and are frequently watertight. They differ from wells in that they can be, and often are, lowered through water to the ground surface. They have been placed in depths of water of nearly 40 feet; whereas, it will be remembered, wells only lend themselves to a start from bare ground. It is the only method by which foundations of extraordinary depth under water can be made, and the deepest foundations in the world have therefore been formed in this way. Notable instances of such foundations are the Hawkesbury Bridge, N.S.W.: extreme depth 155 feet below low water; the Poughkeepsie Bridge across the Hudson at New York: 135 feet extreme depth; and the Sara Bridge, Lower Ganges: 160 feet depth. Open caissons are suited only, except in special conditions, to penetration in soft soils like sand, silt and mud, because dredging is practically impossible in other materials. Stiff clay, boulders, and obstructions of any kind, like trees or existing foundations, call for the aid of divers. Regulation and control of dredged caissons during sinking, especially where obstructions are met with, may be very difficult if not impossible. Plumbing may be managed by pulling or pushing over the head, by increasing the sinking weight or loading on the high side, and by dredging out the ground inside and outside the caisson at the highest point. But one great disadvantage of the method is that the cutting edge is out of sight, and can only be dealt with in a blind way either for undercutting or for packing it where necessary.

The special conditions alluded to are met with in the Thames, the bed of which is London clay. Open caissons on this bottom may be dredged and cleared by divers until sunk about 5 feet into the impervious clay: the water being then pumped out excavations can be proceeded with in the dry. The foundations of the Cannon Street and Charing Cross Railway Bridges, and of the Tower Bridge are examples of this special application of open caissons. The caissons of the Tower Bridge piers formed the walls of a great coffer-dam enclosing the central area from which the water was pumped, and excavation done in the open.

Open caissons are designed usually to meet the same conditions as pneumatic caissons: both open and pneumatic sinking, indeed, are sometimes used at different stages; and the caissons are handled in similar ways. Design will be considered with reference to pneumatic caissons, but some details may be given here. Cylindrical caissons are very common. They are made of steel plates, usually $\frac{3}{8}$ inch thick, in horizontal sections. The joints may have angle flanges bolted together between which an insertion joint can be placed; a similar joint, or a lap joint, may be adopted for riveting. The cutting edge is strongly made with angle ring and thickening plate riveted together with the bottom shell strake. Close riveting and caulking may be adopted throughout so as to secure watertightness, or only at the lower parts, reliance higher up being placed on the concrete for this purpose. Above ground level the cylinder is often of cast iron, because the racking forces from the strata during sinking are absent, and cast iron has a better appearance, resists corrosion better, and is otherwise suitable. Cast-iron cylinders range in diameter from about 6 to 12 feet. The thickness of metal varies from about $1\frac{1}{8}$ inches to $1\frac{1}{2}$ inches and is a matter of judgment. Vertical and circumferential joints should be faced, either over their whole surface or on strips, and bolts, 1 inch to $1\frac{1}{2}$ inches diameter, turned. Sometimes rust cement or other jointing material is used on joints with fitting strips. The horizontal sections of the cylinder may be complete rings or composed of segments. The sizes of the rings and segments are fixed from foundry considerations, or with reference to handling and transport. Certain cylinders of the Clyde Bridge (1870), Caledonian Railway, at Glasgow, were 15 feet in diameter made in complete rings, 6 feet deep, without joints, because the builder preferred to make them without joints. Segments for transport should have the horizontal flanges of the same inside radius as the outside of the cylinders, so that in stacking together they are parallel to each other, and do not bear on the edges only. The depths

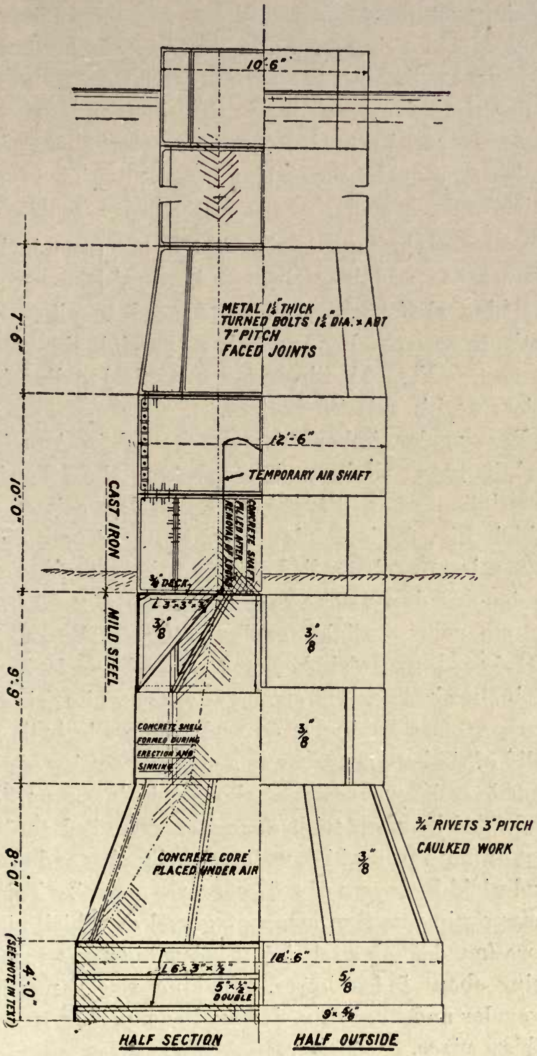


Fig. 21.—Section of typical cylindrical caisson.

of rings run from 5 feet to 7½ feet, and the number of segments in a ring from 3 in a 6-foot cylinder to 6 in a 10-foot cylinder. Strength is provided by the annular ring of concrete inside the shell, and the concrete is tapered at the bottom to meet the cutting edge. This ring provides the sinking weight, but if it is very thick under-grabbing of the cutting edge is difficult. The central shaft must not be less than 3 feet 6 inches diameter to pass the grabs. In large caissons several grabbing shafts are provided, spaced to allow the grabs to command as perfectly as possible the whole bottom. Fig. 21 illustrates a typical cylindrical caisson with splayed bottom.

Rectangular timber caissons are used in America, and the type will be understood from the following description of a Poughkeepsie Bridge caisson of which Fig. 22 is a plan in outline. There were four caissons (also called cribs) of the same general design and dimensions, each being 100 ft. × 60 ft. at bottom, with a slight regular batter to the top, by 104 feet deep. Timber was generally of 12 in. × 12 in. hemlock, except cutting edges of white oak, the lower 20 feet having cross and side walls built up solid of timbers cut away to a splay at cutting edges. Double walls were formed above this lower portion and filled between with concrete. The bed to 100 feet or more below high water was silt, clay and sand, underlaid by layers of a firm coarse gravel. Below these again was firm compact gravel on which piers were founded, about 135 feet below high water, rock being about 5 feet lower. Sinking was somewhat irregular and uncertain, drops of as much as 10 feet taking place.

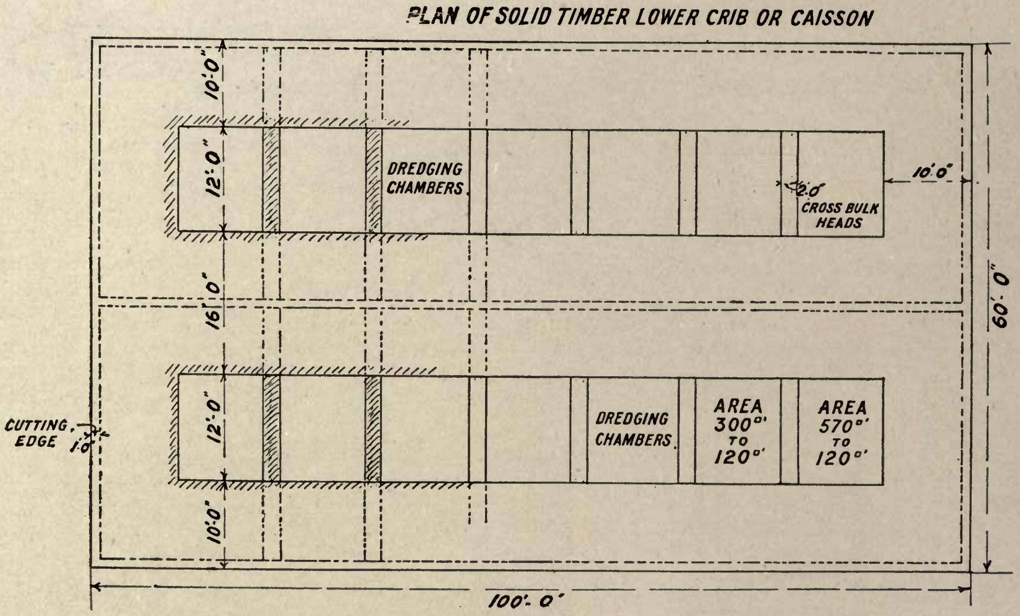


Fig. 22.—Sketch plan of timber caisson for Poughkeepsie Bridge.

Fig. 23 illustrates the concrete caisson used for Pier No. 2 of the American River Bridge, Sacramento, California, of the Southern Pacific Company.

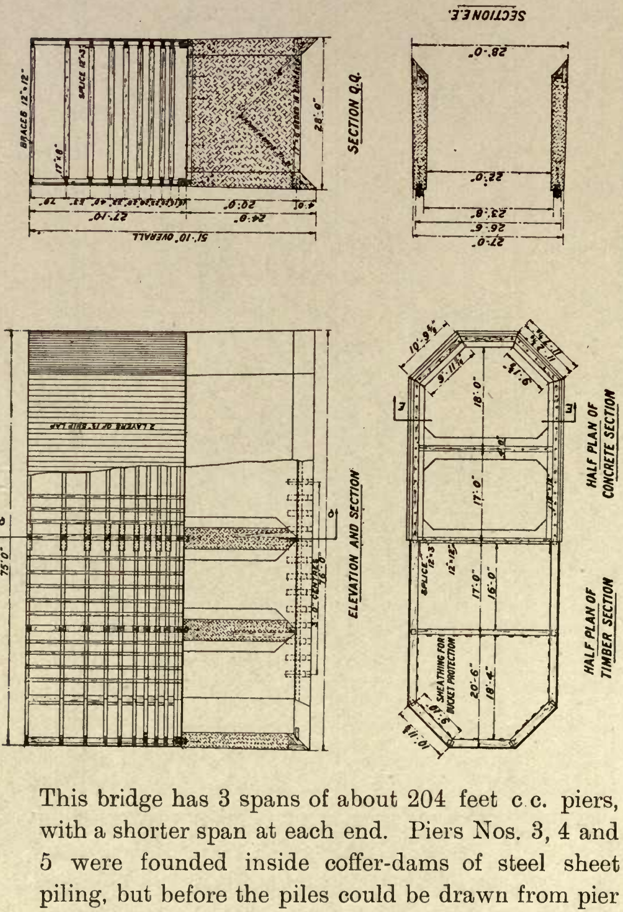


Fig. 23.—Concrete caisson, pier No. 2, American River Bridge, Sacramento, Southern Pacific Company.

This bridge has 3 spans of about 204 feet c.c. piers, with a shorter span at each end. Piers Nos. 3, 4 and 5 were founded inside coffer-dams of steel sheet piling, but before the piles could be drawn from pier No. 3 and re-driven for No. 2, the usual floods came down from the Sierra Nevada Mountains and put them out of reach for some months. In this emergency, as the site of the pier was above water, it was decided to experiment by sinking a concrete caisson by means of open dredging. The concrete was allowed twenty days to set, and thereafter grabbing with orange-peel buckets was started. The sinking was through nearly 50 feet of material, mainly sand and slickens (sediment), with a bed of large gravel and stones at the bottom. Settlement at first was rapid, being as much as 8 feet in ten hours, but towards the bottom it became slow. The last 3 or 4 feet took as many days to sink. Bunkers were built on the top of the caisson and filled with sand, loading to the extent of about 400 tons. Whether or not less loading would have been sufficient was not ascertained. The caisson was stopped short of hard pan or bed rock because the stones could not be removed by the buckets. Six feet of concrete were deposited in the bottom under water, and after four days the water was pumped out and the pier finished in the dry. Leakage was trivial, perhaps not more than 1000 gallons per hour. Several changes would be made in building another pier of the same kind. The cutting edge would be made thinner, stepped out to thicker walls carried higher, and the wells would be made smaller. Wells 10 feet diameter are considered large enough. Table VI gives figures of quantities and costs of this pier.

CHAPTER XII.

PNEUMATIC CAISSONS.

A PNEUMATIC caisson is in effect a heavy hollow structure with a top and no bottom, and the pneumatic process consists in expelling and keeping out the water from the inverted structure by compressed air as it rests on a river bed ; so that the bed thus exposed may be excavated by workmen inside the caisson. The weight of the caisson and everything it carries is opposed to the upward pressure of the compressed air, and the excess of the weight over the pressure must be sufficient to force the caisson down into the excavated bottom. The process is familiar, but it may be understood in principle and outline from the diagrammatic section, Fig. 24, of a circular caisson or cylinder, and the following brief description of the air-lock. The air-lock, through which men and materials have to pass or are passed from the outside atmosphere to the compressed air, or the reverse, is an essential feature of the pneumatic process. It consists of a totally enclosed compartment, having two doors and two air valves, which communicate directly with the caisson and outer air. The inner door opens outwards only, into the caisson ; the outer door opens inwards only, from the outside atmosphere. The inner valve connects the caisson with the compartment, and the outer valve the compartment with the outside. To maintain the pressure one or other of the doors, with its corresponding

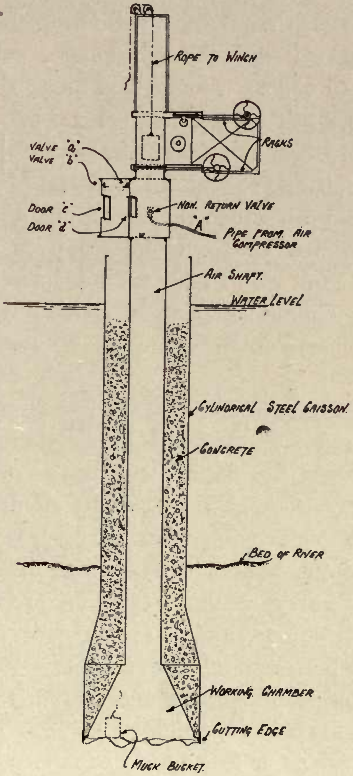


Fig. 24.—Diagram section of cylindrical caisson with air locks in position.

valve, must always be shut. Suppose the inside door and valve are shut, and the outside ones open. The steps to be taken in locking in would be these: Push the outer door close, shut the outer valve and open the inner one: the pressure will rise in the compartment until equal with the caisson, and the inner door will fall open. Locking out: Pull the inner door close: shut the inner valve and open the outer one: the pressure will fall in the compartment until equal with the atmosphere, and the outer door will fall open. The air-lock must be above the water level, so that workmen can get into it should the pressure be cut off and the water rise quickly inside the caisson; and an automatic non-return valve is placed between the air supply pipe and the caisson, so that an accidental stoppage of the air supply may not result in a sudden lowering of the pressure. There must, also, be a safety valve to prevent an increase of pressure above the necessary maximum.

The pneumatic process of sinking caissons for obtaining foundations, because of the certainty it affords that a proper foundation will be secured, is the method now followed in practically all deep foundations for heavy and important structures. The laying bare of the bottom from start to finish makes it possible to carry out the excavation properly, to remove obstructions, and to use means to maintain the vertical descent of the caisson. Further, the bed can be examined and tested, and, when finally approved, the hearting solidly filled in. The greatest depth to which a foundation can be taken by the pneumatic method, having regard to the health and safety of the workmen, is from 100 to 110 feet below low water. These depths correspond to air pressures of 44 to 49 lb. per square inch above atmosphere, and may be reckoned as the present limits at which it is safe (with special precautions) and profitable

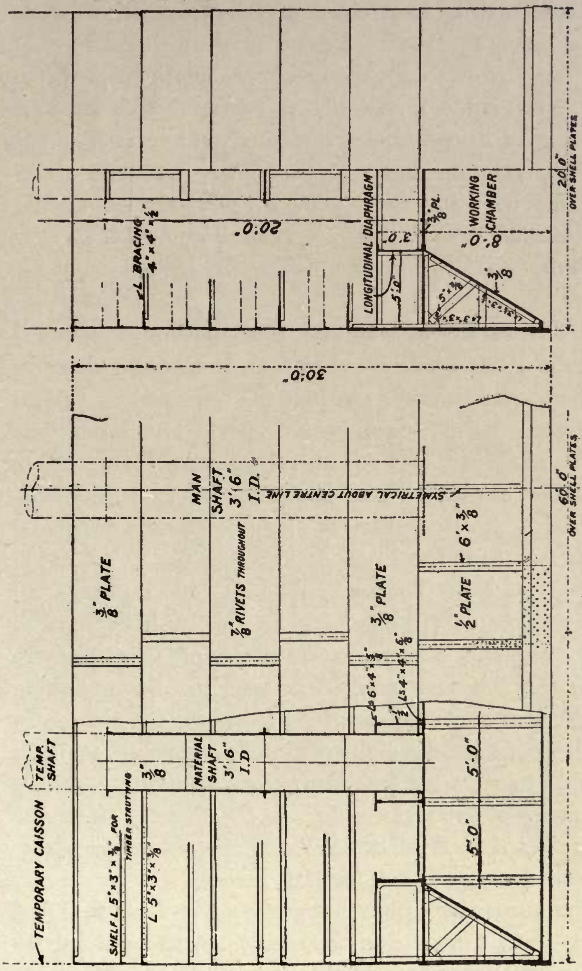


FIG. 25.—Typical rectangular steel caisson.

for workmen to be employed. Eads's St. Louis

Bridge (1870) has foundations carried by the pneumatic method to a depth of 109½ feet below water surface; at Barrow Bridge (1908), Rosslare and Waterford Railway, the maximum depth reached was 117 feet below high water, the maximum air pressure being 44 lb. per square inch above atmosphere; and the maximum depth to which the cylinders of the bridge over the River Towey, Carmarthen (1911), Great Western Railway, were sunk was about 109 feet below ground level, the maximum air pressure being 50 lb.

The variety in size and construction of caissons is too great for any attempt to be made to describe them even briefly, but Fig. 25 represents a typical rectangular steel caisson for a large foundation, and Fig. 26 illustrates the usual American practice of timber caissons. The latter is one of the pier caissons of the Sixth Street Viaduct, Kansas City, and is taken from the Proc. Am. Soc. C.E., Vol. XXXV, p. 81 *et seq.* Timber is abundant in America, and can be quickly obtained, is easily handled and worked, and caissons made of it lend themselves to floating into position. In place, their comparative lightness and buoyancy help to reduce the bed loads. The roof and sides of the working chamber are caulked, and there may be impervious sheeting between courses of the timbers, for air tightness. The main portion of the Sixth Street Viaduct consists of two truss deck spans each 310 feet 10½ inches c.c. of end pins. The three river piers supporting these are founded on rock, timber caissons sunk by the pneumatic process having been used for two of them. The mid-channel caisson was built on shore and floated to site with the aid of a false bottom, while the east shore caisson was built in place. The former reached bed-rock 38·75 feet below low water, and the latter 57·75 feet. High water is 35·5 feet above low

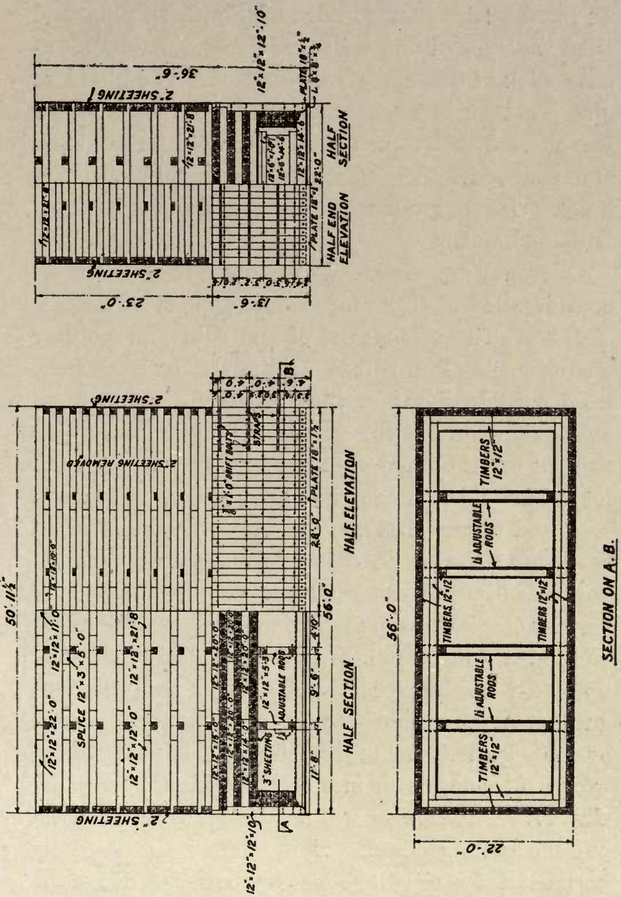


FIG. 26.—Timber caisson, Sixth Street Viaduct, Kansas City.

water. The overlying material was found to vary from soft muck and running sand to clay, firm sand, and gravel. The timbers used for the caisson were 12 in. × 12 in. Oregon fir and Southern pine, and were well bolted and drifted together. Every alternate stick of the upper roof timbers was omitted to give bond for the concrete. The working chamber was sheeted with 3-inch planks, bevelled for caulking, and fastened with boat spikes driven through bored holes. Oakum grummets were placed under heads of bolts and spikes.

Caissons are designed mainly from practical considerations. Take, for example, the steel caisson, Fig. 25. The thicknesses of the skin and working chamber plates are fixed arbitrarily or sometimes from considerations of shear. They are supported against pressure by the concrete filling. The outside skin plates at bottom would be made $\frac{1}{2}$ inch or $\frac{5}{8}$ inch for stiffness, but ^{3} inch is sufficient for the sloping, roof, and upper plates. The cutting edge is made very heavy to resist possible deformation, which might be caused by the caisson being sunk on the top of boulders, or by blasting. The two angles there might be 6 in. × 6 in. × $\frac{3}{4}$ in. and the skin plates thickened up by 2-$\frac{3}{4}$ inch plates. The spacing of the transverse girders may be about 4 to 5 feet. The loads carried by these girders are fairly definite, being due to the wedge of concrete or masonry above, as shown by Fig. 27. The sinking will have a tendency to burst out the cutting edges. This might be resisted by horizontal ties, or will be met by the resistance of the shoe brackets, the stress being greatest at the corner formed by the sloping and roof plates. If the caisson were evenly supported at all times, bending and shearing would be slight, considering the caisson as a beam. But actually there is a possibility of very serious bending especially with a shallow caisson. Suppose a shallow caisson supported on and near its centre only, the tension on the upper edges might be very severe. The other extreme case, of support at each end only, would have the opposite effect. The worst condition

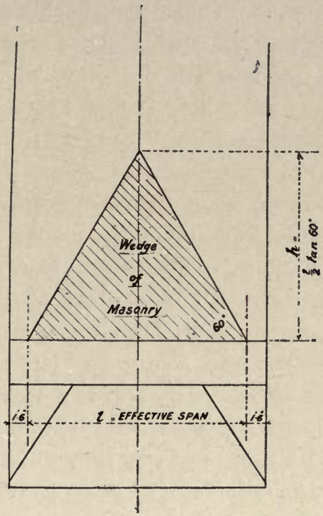


FIG. 27.

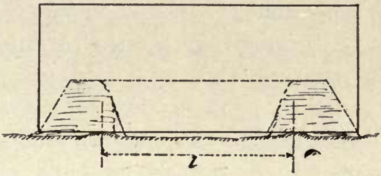


FIG. 28.

may be determined beforehand by arranging that support must always be maintained at certain points, as Fig. 28. Find the greatest bending moment for this condition, and considering each side as a girder and starting with the cutting edge as previously fixed, proceed to find a balanced section which will resist this bending moment. If the caisson is lowered from overhead beams, the bending from this condition must not be neglected. Above the mass concrete over the working chamber buckling of the shell plating and deformation of the whole must be prevented by bracings or strutting. The inside of the working chamber will be caulked airtight, and also usually the outside shell. The riveting will be designed for this tightness and also to develop the strength of the sections. Since a caisson is a temporary structure the allowable stresses may be fairly high, say—

$f_t = 10{\cdot}0$ tons per square inch on net section.
$f_c =$ a maximum of $\frac{8}{10} f_t$ on gross section.
$f_s = \frac{8}{10} f_t$.

A caisson (1) may be built ashore and launched and floated to its site, being there either moored over position, or docked in a fixed surrounding staging previously constructed. In shallow waters the flotation may be temporarily increased by means of a removable watertight bottom, or it may be slung between barges or pontoons for the same purpose. Or (2) the lower part may be built on a timber platform supported on piled falsework, near low water level; and when this part is completed it can be lifted by links, screws, or tackles, hung from overhead girders supported on the falsework, the platform withdrawn from underneath and the caisson gradually lowered.

Erection is continued, and concrete added until it either reaches the bottom or floats. The latter is the usual method adopted for cylinders, there being three points of suspension, at corners of an equilateral triangle, to ensure equal loading on each suspender. Where the depth of water precludes the use of staging and there is no suitable site in the neighbourhood for building and launching, caissons have been built on a temporary platform placed across between two pontoons, and when so far completed, suspended from overhead gear supported from the pontoons, the platform withdrawn, and the caisson gradually lowered as before. If the caisson is floating, it is sunk to river bed by the building up of the concrete or other masonry, and a temporary caisson may be needed on the top to exclude the water from the building until it reaches the bed. If the caisson is suspended until it reaches the bottom, only sufficient concrete will be added as is necessary, in order to keep the weight on the suspending gear as low as possible.

A caisson should be landed on a level bed, although this is not always done; and great care should be taken in placing it correctly. Whether the bed is levelled before the caisson is brought near the site, or afterwards, will depend on circumstances. It is not easy to level carefully an ill-defined area under water; but with a caisson sunk on its bed in a tidal water previous levelling must be done. Sand bags may be used for levelling up, or a margin of sand bags filled between with sand. With caissons suspended until they are placed on the bed, the levelling may be done by divers just when the caisson has reached the highest part of the bed. Working round the edge the divers can level off the higher parts and fill up the lower, the caisson meanwhile being gradually landed. In adjusting a caisson for position, allowance may have to be made for the drifting effect of the current or tide. This allowance will depend on the depth of water and on what adjustments are possible when the caisson reaches the bottom. But once the caisson has entered the bed a tide or current will not have much effect in ordinary circumstances. It is different with a caisson being sunk into a steeply sloping bank, even on a levelled bed. The bank above will exert a steady pressure, tending both to tilt the caisson out of plumb and to shift it bodily as it moves down. No general guidance can be given as to the allowance which should be made. It must be judged independently for each particular case.

CHAPTER XIII.

PNEUMATIC CAISSONS (*Continued*).

THE form the excavation in the working chamber should take depends on the nature of the material, and partly on the amount of the sinking weight—that is the excess of the downward load over the uplift due to the air pressure. The ideal condition is that in which the sinking weight is sufficient to cause the caisson to move down gradually as the material is removed, without any reduction of the air pressure. This is possible only in soft or comparatively soft soils in which the cutting edge would never really be bared, and with this in view as much permanent loading should be provided as possible. In a case of this kind the bottom may be taken down in thin layers, level all over, or excavation may be confined to the central area the descent of the caisson pushing the soil inwards. Trouble may be experienced with blows when the air pressure is lowered, especially if the holding-up of the caisson is caused by upper strata while it is passing through a quick or running bed. In such a case the central area should be kept as high as possible and the digging taken out in the form of a trench round the edge; with the idea of choking off and opposing the incoming material as the caisson is lowered. In firmer ground the cutting edge must be undercut to a greater or less extent, and the pressure lowered, or in extreme cases taken entirely off, to cause sinking. On an irregular bottom there is often a tendency to tilt or cant to one side or other, and banking and timber packings may have to be used to counteract this tendency. A caisson may be seated several feet into rock, the rock being removed by blasting. The principle in this case is to weaken the centre by blasting upwards, and then blast the edges inwards (see Fig. 29.)

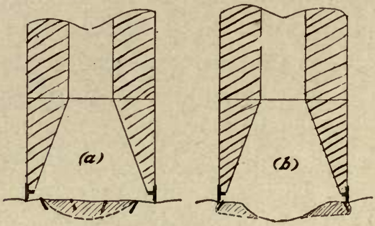


Fig. 29.

Caissons get out of plumb, or of position, or both, either from carelessness or from unavoidable causes, and have to be righted. These errors occur usually in smaller caissons: in very large caissons special precautions are taken to prevent them. Large caissons are easier to control than small ones, and caissons with parallel sides than caissons with outsplayed bottoms. If the bottom parallel part of a splayed circular caisson is less in depth than 1 to 1½ times the diameter, considerable difficulty will usually be experienced in sinking it plumb. If this rule were adhered to in the circular caisson, Fig. 19, the bottom part should be not less than 18 feet deep instead of 4 feet. Righting in any case is an interruption to the run of sinking and costs money, and may at times be very expensive, and errors should be anticipated and prevented if possible. In the first place the caisson should be landed in correct position as plumb as possible. It should then be plumbed exactly by the preliminary digging and thereafter carefully surveyed. If much out of position the best way may be to lift it and pitch it again, but correction for a few inches in any direction can usually be made by tilting the head, sinking in an inclined position, and then plumbing when the desired position has been reached. This lateral movement may be assisted by the use of rocking shores inside the working chamber, Fig. 30, or by steel sliding wedges or steel stakes placed as shown in Fig. 31. Plumbing may be done by undercutting at the high side and by holding up with plates and wood packings inserted under the low side, assisted by pulling and pushing over the head. Loading may be added outside at the high point, and the ground outside may be weakened by dredging. Corrections should be made at first in order to

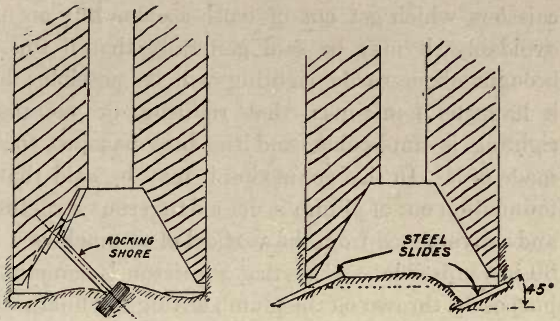


FIG. 30. FIG. 31.

Methods of adjusting caissons for position.

secure a fair start, and extreme care should be taken in sinking for at least the first 10 feet. After that the likelihood of errors is less. It sometimes happens through inattention, or because of delay in making corrections, that a caisson is wrong at a considerable depth in the ground. The before-mentioned methods can be used, successfully in many cases, but unsuccessfully in others for no apparent reasons. With caissons which get out of truth accidentally or unavoidably, it may be said generally that if this is brought about easily righting will be possible; but it happens sometimes that righting or complete righting is impossible, and the best has just to be made of it. In this connexion it may be said that a foundation out of plumb is not a dangerous structure, and a divergence from the vertical of as much as 1 in 60 is admissible. Also, that a caisson is sometimes purposely thrown off the plumb during the final stage of sinking in order to bring the head nearer true position. The banking up of the spoil at one point outside a caisson is often the cause of it being thrown out of plumb. Like the sloping bank mentioned above it exerts a steady constant pressure in one direction.

In large timber caissons it is necessary to support the caisson evenly at all stages of sinking in order to prevent damage to it. This as well as level sinking was ensured in the uebec Bridge caissons by means of sand jacks supporting them all round, lowered simultaneously by a common exhaust. At the King Edward Bridge, Newcastle, two broad flange girders crossed the working chamber at intermediate points. In excavating these were left supported by banks until the caisson was otherwise cleared, and level lowering accomplished by gradually picking out the two banks together.

The special plant required for pneumatic sinking includes the compressor, the air pipes and valves, the man-locks, and the material locks with their hoisting gear. The capacity of the plant should be ample, and all the plant should be of the best both in design and condition. The risks are great enough in air sinking, as in other foundation methods, without increasing them by the use of inadequate plant. The capacity of the compressing plant will be decided by a consideration of the highest pressure to be attained and the greatest area of foundation to be taken at one time; and it must be assumed that the bed is of open material and will allow the air to escape freely. There is a minimum quantity of air which should be supplied to each sinker, say 3000 cubic feet per hour, for proper ventilation, but the capacity arrived at as above should be more than equal to this. In close or sealed material, the working chamber must be ventilated by pipes taken to the surface.

Where interruption of the sinking would be attended with danger, or where delay would be serious, the compressing plant should be in duplicate, so that in the event of one set breaking down the other set would be available. It is not meant that the spare set should be ready to be switched on at a moment's notice, as is the intention of some specifications on this point. Sudden breakdowns with compressing plant kept in good order are so unlikely as to be outside of practical consideration. But almost constant overhauling is required where plant is running continuously, and the sets in duplicate afford facilities for this overhauling. Spares of air valves, especially, and wearing parts should be kept ready, and always replaced by fresh spares when put into use. If the plant is steam-driven, the weakness of the boiler is the feeding arrangements. Whether injectors or pumps are used there should be two to each boiler, with a spare in readiness. The size of the air main will depend within limits on the size of the compressor, the velocity of the air being greater or less as the diameter of the pipe is smaller or larger. If the distance is very great from the compressor to the caisson the diameter should be somewhat increased.

The air is heated due to compression and to friction in the pipes and it must be cooled, and should be dry, before it enters the caisson. The flexible pipe to the caisson is of spirally wound copper and should be very carefully guarded, the breaking of it, and the failure of the boiler feed being, perhaps, the two likeliest causes of accidental stoppage.

Air shafts in a caisson are about 3 feet 6 inches to 4 feet 6 inches diameter. Where there is one shaft only, both man- and material-locks will be placed on the same shaft, the latter on the top. Where there are several shafts, the man-lock might have a separate one. Man-locks for bridge caissons are much too small, particularly where high pressures are attained and men are compelled to remain in them for some time. This defect can hardly be avoided, perhaps, owing to limited room. The usefulness of a material-lock is measured by the amount of material which can be passed through it in a given time. Material-locks are of two kinds—locks through which the full muck skip is brought out every lift, and locks in which the spoil is tipped from the skip into a spoil chute inside, the chute itself being a lock. The hoisting is done preferably by electricity; where steam is used a flexible steam pipe has to be provided from the boiler; and hoists are sometimes air-driven. The Arrol lock has been in use, with little modification, for thirty years (Fig. 21). It is of the former type, and a skip of $\frac{1}{3}$ cubic yard capacity can be passed through it every 2 to 3 minutes for the round trip, including time taken to fill, hoist about 50 feet, empty, and lower the skip again. Illustrations of many different locks will be found in the Proc. I.C.E. and Trans. Am. Soc. C.E.

Spares for all lock gear, door rubbers, etc., should always be at hand, in the same way as for the compressing plant.

CHAPTER XIV.

THE EFFECTS OF COMPRESSED AIR.

The pneumatic method of obtaining foundations is of very great importance to engineers. In many cases it is the only method applicable, and in others it is the most preferable. And its importance is just as great in subaqueous tunnelling. As the success of the process depends on the ability of the human body to withstand the effects of the compressed air, it is necessary to learn to understand as far as possible these effects, and the prevention and treatment of them. In the earlier days of pneumatic sinking and tunnelling, caisson illness in the higher pressures was very frequent and cases of death not unusual, but where care compatible with existing knowledge is taken these evil results are very greatly reduced. Our present knowledge has been gained both from experimental research, and from careful observations made, and improvements introduced, on actual works. The names of Professor J. S. Haldane, Dr. Leonard Hill, and Mr. E. W. Moir, M.Inst.C.E., should be mentioned in these connexions. It is incumbent upon anyone who is responsible for work of this kind to make himself acquainted with the literature of the subject, and reference may be made to the Admiralty Report on Deep-Water Diving (1907), Dr. Haldane's paper read before the Royal Society of Arts in November, 1907, and Dr. Hill's paper read before the same Society in March, 1911 ; besides various papers read before the Institution of Civil Engineers and the American Society of Civil Engineers, with the discussions thereon.

Caisson disease or compressed-air sickness occurs only after decompression, and is due to bubbles of nitrogen appearing in the blood and interfering with the circulation. Pain, paralysis, or death, may result, depending on the place where the circulation is stopped. When a gas is brought into contact with a liquid, the latter takes up the gas in simple solution, until a state of saturation is reached. The amount taken up depends on the solubility of the gas in the liquid, and on the temperature, and varies directly with the pressure in accordance with Dalton's Law. When a man enters compressed air, the ventilation of the lungs keeps the CO_2 present at a fixed percentage without difficulty ; and the blood being practically in contact with the air while passing through the lungs takes up an increased amount of nitrogen and oxygen. The oxygen is dealt with by the hæmoglobin of the blood, and the nitrogen, which forms by far the larger proportion (79 per cent), remains unaltered. The blood every time it circulates through the lungs probably gets completely saturated, in its passage through the body it disposes of part of the acquired nitrogen, and when it returns to the lungs it is again completely saturated. The process goes on, and in time for any given pressure the blood and tissues of the body are alike completely saturated. The rate of saturation decreases gradually as the relative saturation of the outgoing blood and the tissues decreases. It may be represented graphically

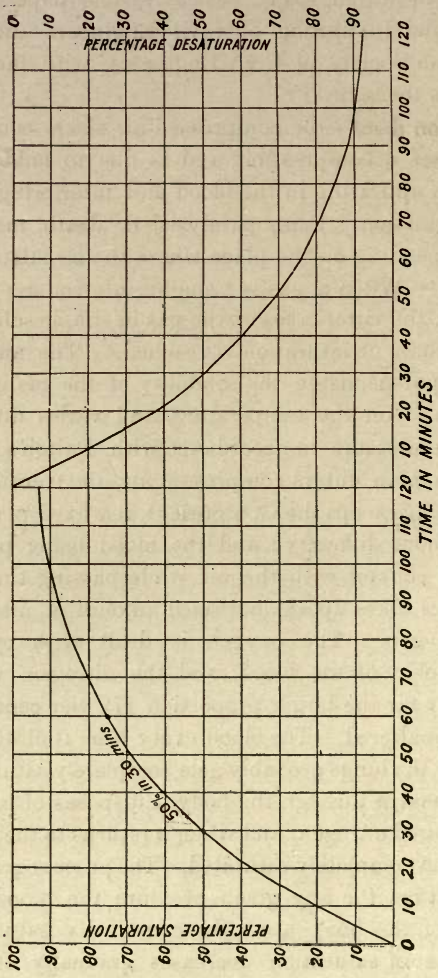


Fig. 32.

by the logarithmic curve on the left-hand side of Fig.

32, taken from Dr. Haldane's paper, assuming that compression is instantaneous. Every part of the body is not saturated in the same time, because there are slowly saturating parts like fat, and parts quickly saturated. Fat absorbs as much as five times the nitrogen taken up by the watery tissues. Several—perhaps three or four—hours pass before every part is completely saturated.

On reduction of the air pressure, saturation of the blood and tissues will become supersaturation, and the excess nitrogen will tend to free itself in the form of bubbles. Actually owing to the viscosity of the blood, these bubbles are not formed until a certain amount or proportion of supersaturation exists, and if the period of decompression is extended long enough, desaturation will take place, except for a certain lag, in a similar gradual and safe way to saturation, as indicated by the curve on the right hand of Fig. 32. If the time of work in compressed air is curtailed so that saturation is incomplete, the time for safe desaturation will be shortened proportionately. Paul Bert, an early investigator, from experiments concluded that at least 12 mins. per atmosphere should be allowed for decompression from higher pressures, and others have recommended 20 mins. for pressures over + 30. Taking the latter figure, with + 35 lb. pressure, the time for decompression would be $\frac{35}{15} \times 20 = 47$ mins. Table VII gives lengths of shifts and decompression periods as suggested by Drs. Hill and McLeod (see Proc. I.C.E., Vol. CLXV, p. 237). These allowances have been much reduced in practice without disastrous results. At 15 lb. gauge 3 mins. may be allowed for uniform decompression, at 25 lb. 6 or 7 mins., and at 30 lb. 14 or 15 mins. may be allowed. At King Edward Bridge, Newcastle, 1 min. was allowed for every 5 lb., or 3 mins. for every atmosphere. Up to 22 lb. pressure the men worked ten hours each day in three shifts with the usual meal hours. Between 22 lb. and a little over 30 lb. (the maximum) the meal intervals were increased by half an hour, and the total time worked per day was eight and a half hours. During the construction of the double tunnel under the East River, New York, by Messrs. S. Pearson & Co., Ltd., the pressure was 34 lb. and the men worked two three-hour shifts daily. The time normally allowed for decompression was 15 mins. or 2¼ lb. per min., and in four months (1906) the deaths amounted to one in every 15,000 decompressions.

What is known as stage or the Haldane method of decompression was introduced by the publication of the Admiralty Report, cited above, in 1907. The times required for uniform decompression from the higher pressures—say from + 32 to 45 lb.—are impracticably long and do not prevent frequent illness and occasional deaths. The actual difference in relative pressures may be a dangerous amount even after long periods, because of the lag in desaturation. The stage method reduces the time required by hastening decompression at the beginning, and is explained in what follows. It is a matter of common experience in sinking that rapid decompression from + 19 lb. (at the least) is for all practical purposes unattended with risks. The ratio of + 19 lb. to atmosphere is 2·3 to 1. Arguing from this, and because the volume of gas capable of being liberated on decompression to any given pressure is the same if the relative diminution of pressure is the same, Dr. Haldane concluded that if the relative diminution of pressure did not exceed 2 to 1 there would be no danger of bubbles forming by rapid decompression; that is to say, the danger of rapid decompression depends not on the absolute difference between the initial and final pressures, but on the proportional difference between the two. Experiment practically confirmed this conclusion. Rapid decompression (one stage), therefore, is possible down to half the absolute maximum, and the pressure is then lowered gradually at a rate calculated to reach atmosphere when the nitrogen in the body is at + 19 lb. This is the second, or slow, stage. The rapid stage may be timed at 3 minutes. Table VIII shows rates of decompression recommended by Dr. Haldane (Journal of Royal Society of Arts, Vol. LVI, p. 225). It has taken into account that men will usually come out for meals after periods of three hours' work, and since they will not have time to desaturate during the interval, they will be more highly saturated after a second or third period of work than after the first. Figures for continuous exposure, also, are given. Example of the use of the Table: Pressure + 30 lb., or gauge pressure: half maximum = $\frac{30 + 15}{2}$ or 22½ lb. or 7½ lb. by gauge; 7½ × 5 = 37½ mins. for second stage, or 3 × 37½ for the two stages after three hours' work; 3 + (7½ × 9) = 70½ mins. The times, it will be seen, are very long, but are calculated to give the same safety for work over 20 lb.

pressure as exists with work under 20 lb. Doubtless the margin for safety is too great. It will be seen that the best result would be arrived at if the rate of decompression in the second stage was varied or regulated so that the curves of desaturation and decompression were kept parallel. The stage method was adopted at the Elbe Tunnel Works with a slight advantage as compared with uniform decompression which was also tried.

Since the cleansing of the blood and tissues during decompression is carried out by the circulation of the blood, desaturation will be hastened by exercise during decompression, because it will quicken the circulation. This point is brought out in Dr. Leonard Hill's paper cited at the beginning of this chapter (Jour. Roy. Soc. of Arts, Vol. LIX, p. 399), and beneficial results of stage decompression, plus exercise during pauses between the stages, were experienced on the East River Tunnels already mentioned. A clear account is given in Mr. Henry Japp's paper in Vol. XXXV, p. 378, Proc. Am. Soc. C.E. These anticipated in some degree Dr. Haldane's theoretic stage decompression. Towards the end of 1906, at a later period than that at which the figures given for uniform decompression were obtained, a second diaphragm was put in one of the tunnels, and the pressure between the two diaphragms lowered to 15 lb. The number of cases of disease was very small for that tunnel, the exercise of walking at low pressure between the two tunnels seemed to help desaturation. The procedure was : 35 lb. at face to 15 lb. between diaphragms quickly, walking to second diaphragm 5 mins., 15 lb. to atmos. in 10 mins. : in all about 16 mins. When maximum pressure was less than 32 lb. an eight hour shift with half hour meal interval in 15 lb. was adopted Some time after the publication of Dr. Haldane's paper it was necessary to raise the tunnel pressure to 40 lb. gauge. There were 3 diaphragms and sets of locks. Pressure at face 40 lb., intermediate chamber 29 lb., outer chamber 12½ lb. Time taken first lock 5 mins., inner chamber 10 mins., second lock 8 mins., outer chamber 10 mins., third lock 15 mins.—in all 48 mins. It was calculated that at the end of this time the equivalent pressure in the blood would be 27 lb. instead of 19 lb. which form the basis of Dr. Haldane's figures; 27 lb. was adopted because in most compressed-air works very little trouble is experienced up to that pressure. Three hundred and thirty seasoned men were employed in these conditions for thirty-six days, working three hours on, three hours off, and three hours on, and no severe or fatal cases of illness occurred, while very little time was lost through caisson disease.

Breathing oxygen to hasten decompression, by Fleuss apparatus or otherwise, has been recommended. Oxygen combines with the nitrogen and helps to clear it out.

Compression, or locking-in, is unattended with danger. If it is performed too rapidly, pain, which may be very acute, is experienced in the ears, due to the difference of pressure on each side of the drums. Relief may be got by the action of swallowing, and the time may be regulated by the feelings of the persons going through the lock.

Caisson disease manifests itself usually within an hour after decompression, although the interval may be longer. The most common symptoms are pains in the limbs—frequently the knees, and pains in the abdomen. The pains in the limbs are known as bends and may be so acute as to cause strong men to writhe in agony. There may, also, be partial paralysis and convulsions, and the rapidity with which these symptoms show themselves is a measure of the severity of the illness. All the symptoms are consistent with the theory of the disease that bubbles form after the difference in relative pressures has reached a certain amount; and because the bubbles tend to expand as this difference increases, the pains may increase in severity. Instant recompression, which drives the bubbles again into solution, followed by slow decompression in the great majority of cases will give immediate relief and cure the disease. Where recompression has been delayed, or immediate injury has been done to any part, recompression may give relief. The recompression, or medical air-lock, was introduced over twenty years ago by Mr. E. W. Moir on the Hudson Tunnel Works from the experience he had gained on the Forth Bridge. The workmen there, as they do elsewhere, found relief by going back into the caisson. It helped to reduce the percentage of deaths on the Hudson Tunnels from 25 to 1½ per annum, and has saved many lives. A medical lock is a means of dealing with individuals which the precautionary measures of limited shifts and regulated periods of decompression have failed to keep immune. It consists of a steel chamber, with a lock entrance, properly warmed and ventilated, a couch being provided on which the patient may rest. Relief from muscular pain may be obtained from rest in bed and warmth, and by rubbing.

There is no reasonable doubt that air sickness is due to the formation of bubbles of nitrogen as stated, that the risk increases rapidly with the pressure, and that in ordinary circumstances limitation of shifts and lengthened periods of decompression help to minimize the risk to a practically negligible quantity.

There are, however, contributory causes to the disease. The presence of carbon dioxide (CO_2) in undue amount is one of these, not so much in itself perhaps as that it indicates bad ventilation and the evils resulting therefrom. It may be due to impurities in the air supplied, to the burning of unsuitable oil for lighting, and to its presence in the material being excavated, and arises, of course, from the respiration of the workmen. At the Rotherhithe Tunnel, the L.C.C. stipulated that the amount of CO_2 should not be allowed to exceed 1 part in every thousand, but this may be an expensive maximum, and general experience shows that 2 parts in every thousand are not harmful. Ventilation and purity of the air are secured by supplying an adequate amount of air per man per hour. This may range from 2000 up to 6000 cubic feet, and has reached even 15,000. Anything above 5000 is extraordinary, and doubtless unnecessary. Impurities sent over with the air may come from the outside, or may be due to unsuitable lubrication of the air-compressor cylinders; both may be removed, as can also those due to improper lighting. The air should be dry and of a comfortable temperature. There is a fall of temperature and moisture is formed during decompression, and ill effects of this may be prevented by providing warm clothing and a warm hut, with hot coffee, to the workmen, immediately following decompression. It may also be possible to warm and ventilate the lock. No work on an empty stomach, and the avoidance of fat foods are advocated, and, also, abstinence from alcoholic liquors, especially beer.

Workmen must be submitted to a medical examination before employment under air in order to eliminate the weak and unsuitable. Men with weak heart or lungs, or subject to catarrh of the nose and throat, should not be employed (Sir Thomas Oliver, M.D.). Men over forty years of age, and men of heavy or fatty build under that age should be excluded. Table IX gives the experience of Dr. Snell, the medical officer employed by the L.C.C. at the Blackwall Tunnel (1892) with regard to age. Some sound men are found to be susceptible to the disease, while others are practically immune in severe conditions. The susceptible may be discovered by working in preliminary short shifts, but even seasoned men are sometimes attacked. At the Greenwich Footway Tunnel (1901) 13·9 per cent of the men presenting themselves were summarily rejected by the medical examination, and 5·7 per cent subsequently (H. C. Copperthwaite); at the East River tunnels the percentage of rejections was 10; and at the King Edward Bridge, 4.

It may be gathered from what has been said that a great deal of the experience available has been gained in tunnel works, and this is natural because there great numbers of men are employed for long periods at pressures which vary only slightly according to the height of the tide. In bridge work, on the other hand, with each foundation the pressure starts at a low minimum, gradually increasing to the maximum, there being of course daily fluctuations from the tide. There is the further fact that sinking proceeds in stages interrupted by stoppages for further building of the caisson. It is possible, on the other hand, to take precautions in tunnel work inapplicable in bridge work. The risks of caisson sinking by means of compressed air, however, must not be exaggerated. In an experience extending over twenty years, with pressure ranging up to over 40 lb., the author only recollects one death occurring, and although there were many serious cases of "bends," he has subsequent knowledge of only one case of permanent injury. The general practice on works under his control was to select healthy, well-behaved men, to the young side, and all the possible precautions known at the time, stage decompression among the rest, were adopted. It may be repeated that up to 25 lb. by gauge no special risk is attached to work, although precautions should not be neglected, that at 28 lb. the pressure is approaching the dangerous limit, and with 30 lb. and over work is attended with danger and the risks increase very rapidly.

CHAPTER XV.

CONCLUSION.

OTHER methods of obtaining foundations than those described in the preceding pages, or modifications of them, have been used, and brief mention will now be made of a few of these.

Foundations which depend wholly, or almost wholly, on their buoyancy are used in special circumstances. One of the most notable instances of the former is to be found in the Howrah Bridge, across the Hooghly at Calcutta, and another is the bridge across the Golden Horn at Constantinople. The Hooghly at Calcutta has a width of about 1500 feet and has a bottom of very loose silt of great depth. The water is not of extraordinary depth, but because of the swiftly running ebb and flow, the latter often with a bore or aeger, the violent scour prohibits the use of piers. Except for fixed abutments, the bridge is carried on floating foundations, or pontoons, which rise and fall with the tide. The bridge has served its day, and among the new designs put forward one by Messrs. Head, Wrightson & Co., Ltd., of Thornaby-on-Tees, has patent submerged pontoons anchored under low water vertically to the bottom. At low water the pontoons carry their maximum load and the tension on the anchor ties is at its least, as the tide rises the buoyancy increases and the pull on the ties increases likewise. The roadway of the bridge remains at a constant level at all states of the tide, while with surface floating pontoons it would rise and fall 10 feet from mean position. The bridge at Constantinople has a length of 1530 feet between abutments, and is 82 feet between parapets. The maximum depth of water is 130 feet, and the substratum to a depth of 230 feet on both banks is silt. There is no tide to complicate the conditions, but except for the abutments, with water so deep there was no choice but pontoons. Pontoon bridges have been used in Bengal over river beds of flat alluvial sandy soils. The beds sometimes change position, and are liable to floods and serious scouring, and pontoons have been used because of their comparative cheapness.

The pontoon foundation which does not carry quite the full load never rises out of position, the excess weight being carried on, and the whole steadied by, fixed supports of some kind. Two single-arm swing bridges, about 112 feet long, were built on foundations of this kind over the River Weaver at Northwich, about 1898. These bridges are on the subsiding area of the salt district, and the problem was to select foundations which could be easily adjusted should subsidence take place. The engineer, Mr. J. A. Saner, M.Inst.C.E., adopted this type. Each bridge has a circular pontoon of 31 ft. diameter × 15 ft. 6 in. deep, composed of four separate segments, rigidly connected to the bridge under its centre of gravity. The pontoon turns with the bridge inside a circle of screw piles carrying the roller path on which the small excess weight is carried. Adjustment for subsidence can be made by screw tops on the piles, and the pontoon always takes the same level relative to the path. The adjustment can be assisted by water ballast pumped from the pontoon. Two small swing bridges to cross passages of the Spencer Dock, Dublin, were built about 1870, on pontoons which carried 95 per cent of the weight, the remaining 5 per cent (5 tons) being carried upon a footstep under the centre of the pontoon.

It is not unusual to enclose an area by piling, dredge out and deposit concrete under water until the water surface is reached, or up to a certain level under water, and then pump out and build in the dry. The latter is in reality a special type of coffer-dam, the essential difference being that the foundation block is placed under water. The foundations for the piers of the large arch bridge over the Rhine at Bonn (1898) of 188 m. clear span, were constructed in this way. Each of the two piers has an area of 32 m. × 13·75 m., with a thickness of 5 m. The top of the concrete coincides with the river bed, and is 5·08 m. below low water. The dam consisted of an inner wall of iron sheet-piling, of H sections, and an outer wall of timber piling 4 m. clear. Before the dam was closed a floating dredger was set to work and the spoil filled between the double walls: afterwards two grabs completed the dredging. The concrete was deposited through tubes in water of a maximum depth of about 10 m., the bottom of the tubes being kept on the concrete surface and raised gradually as the surface rose. Concreting under water must be done with extreme care, and the concrete should be richer in cement than usual to allow for some being washed away. Cement slurry will be found on the surface of the concrete, often of fair depth, but it appears worse than it really is and the concrete after setting will be found hard under the slurry.

Simplex and Compressol piles are suitable for abutment foundations. The former consists of a concrete column placed inside a metal tube, which has been driven previously, the metal tube being gradually withdrawn as the concrete is deposited and rammed. The tube has a closed end for driving, which is opened out before concreting commences. One abutment of the Beaver Bridge over the Ohio River was founded on Simplex piles, placed 3 feet centres. There were 315 piles in the abutment, each 16 inches diameter taken 8 to 10 feet into the gravel substratum. Each pile was designed to carry 35½ tons. A tested pile actually carried 60 tons for 72 hours with a settlement or compression of only $\frac{9}{64}$ inch which was almost wholly recovered after the load was removed. The Beaver Bridge has a total length of 1409 feet c.c. end pins, and a channel cantilever span of 769 feet c.c.

Compressol piles were used for the foundations of the abutments of the Risorgimento Bridge, Rome (1911). This bridge has a single arch span 100 m. (328 feet) clear. The bed of sand and clay was underlain by soils which decreased in value until mud was reached. M. Hennebique recommended Compressol foundations. The ground under each abutment covering an area of about 5520 square feet was compressed by means of 72 concrete piers or piles formed by ramming deep holes by means of falling weights, the holes being afterwards filled with concrete rammed in a similar way.

It was indicated in Chapter II that permanent protection of piers, liable to be damaged by scour, is often provided by enclosing piling. The area of a foundation, also, which has in course of time become overloaded can be spread by the same means. Steel sheet piles are admirably fitted for this work both of retaining and spreading. In cases where the masonry or concrete rests directly on the earth, the piling is driven well down around the footing course, and the space between the footing and steel piling, and under the footing where necessary, carefully packed with fresh concrete. The steel piles are left in position permanently as a revetment and reinforcement. In cases where there is a sub-foundation of bearing piles, already scoured, the coffer-dam formed by the steel piling can be pumped out, and fresh concrete packed round the piles and upwards round the footing. The permanent use of steel piles raises the question of corrosion, and on this point Mr. C. C. Conkling, M.Am.Soc.C.E., expresses the opinion that, in general, steel sheet piling driven below the atmospheric line in earth or water is not subjected to serious corrosion, except in special cases where an exceptional amount of corrosive elements are present; and that two or more coats of good preservative paint will answer in all but these exceptional cases.

The development of foundation engineering in different countries is outside the intention of this book, but the following notes of some bridge foundations carried out by The Foundation Company, New York, are of interest as indicating in some measure American practice. It will be noticed that open sinking was adopted in six of the seven examples, and pneumatic in three only. The allowed load on piles on all these bridges was 15 tons.

1903. *Bridge over Missouri River, Omaha, Nebraska.* Two draw spans each 552 ft. 6 in., eleven girder spans average 53 ft Pivot piers, masonry on concrete, steel caissons 40 ft. 2 in. diameter. Three end piers, masonry on concrete, timber caissons 16 ft. × 46 ft. Other piers on piles. Bed—fine silt, coarse silt, sand, sand and gravel, gravel and boulders. Sinking—pneumatic and open. Base of rail to low water 31 ft., depth of water 13 ft , low water to high water 12 ft. Pivot piers on rock 123 ft. below low water. Excavation—19,300 cubic yards.

1907. *Mississippi River Bridge, Clinton, Iowa.* One draw span 463 ft.; one Pratt truss 186 ft., one 204 ft., eight 150 ft.; eleven girder spans 50 ft. Fourteen masonry piers on concrete, ten pairs 8 ft. diameter steel cylinders on piles, filled with concrete, six masonry and concrete abutments on rock, pivot pier octagonal caisson timber 46 ft. 6 in. diameter, ten coffer-dams 24 ft. to 64 ft. Bed—sand, gravel and shells, disintegrated limestone in sand, sand with clay, limestone. Sinking—pneumatic and open. Draw span, base of rail to low water 29 ft. 8 in., depth of water 18 ft. 3 in., low water to high water 20 ft. 3 in. Low water to rock under pneumatic caissons 62 ft. Excavation—33,100 cubic yards.

1909. *Bridge over Rio Grande River, Brownsville, Texas.* One draw span, 227 ft., two Pratt trusses 127 ft. Three concrete piers, two concrete abutments. Pivot pier octagonal timber cofferdam-40 ft., two coffer-dams 43 ft. × 23 ft., two coffer-dams 32 ft. × 17 ft. Bed—10 ft. quicksand, red clay below. Open sinking. Base of rail to low water 23 ft., depth of water 8 ft. Low water to high water 7 ft., low water to foundation bed 19 ft., piles below. Excavation—1700 cubic yards.

1911. *Kaw River Bridge, Kansas City, Missouri.* One Pratt truss 196 ft., three 180 ft. Three concrete piers, two concrete abutments. Timber coffer-dams 23 ft. × 68 ft. 6 in. Bed—black mud, fine sand, blue clay and shale, coarse gravel, clay and gravel, shale and soapstone. Foundation bed—soapstone and sandstone. Open sinking. Base of rail to low water 35 ft., depth of river 8 ft. Low water to foundation bed 28 ft., 40 ft. piles below. Excavation—11,000 cubic yards.

1912. *Bridge over Pitt River, Vancouver, B.C., Canada.* One draw span 280 ft., two Pratt trusses 255 ft., two 175 ft., nine girder spans 80 ft. Thirteen concrete piers, two concrete abutments. Pivot pier, timber coffer-dam 60 ft. × 37 ft., three coffer-dams 36 ft. × 72 ft., nine do. 23 ft. × 57 ft. Bed—sand and gravel, on which foundations laid. Open sinking. Base of rail to low water 19 ft., depth of water 55 ft., low water to foundation bed 85 ft. Excavation—16,600 cubic yards.

1913. *Bridge over Miramichi River, Newcastle, Prov. New Brunswick, Canada.* One draw span 231 ft. 6 in., two Pratt trusses 275 ft., one 278 ft. Four concrete piers, two concrete abutments. Timber coffer-dam pivot pier 42 ft., octagonal, other piers 51 ft. × 26 ft. Piles under piers. Bed—fine silt,

sand and clay. Open sinking. Base of rail to mean tide 13 ft., M.T. to bed of river 35 ft., M.T. to foundation bed 52 ft. Excavation—4900 cubic yards.

1913. *Bridge over Muskingum, River Coskocton, Ohio.* Two Pratt trusses 239 ft., one 241 ft. Two concrete piers and two concrete abutments. Timber pier caissons 19 ft. 6 in. × 55 ft. Bed—sand, sand and gravel, foundation on gravel. Pneumatic sinking. Base of rail to low water, depth of water 12 ft., base of rail to high water 6 ft., low water to foundation bed 39 ft. Excavation—7000 cubic yards.

TABLE I.

Allowable Net Pressures on Foundation Beds in Tons per Square Foot.

Alluvium and silt	½ to 1
Sand, wet	1
,, fine, dry, compact	2
,, coarse ,, ,,	4
Gravel, fine compact	3 to 4
,, hard, deep-set	6 to 7
Coarse sand and gravel, firm	5
Clay, dry, ordinary	3
,, sound, yellow, dry	4 to 6
,, solid, blue	5 to 7
Shale, firm, protected from weather	6 to 8
Clay and sand	3
Chalk, hard	6
Rock, soft, friable	3 to 5
,, ordinary	10 to 15
,, hard	25 to 30

Consult Newman's "Cylinder Bridge Piers" (Spon), and Corthell's "Allowable Pressures on Deep Foundations" (Wiley).

TABLE II.—SKIN FRICTION.

Bridge.	Foundation.	Embedded Depth in Feet.	Average Skin Friction in cwts. per sup. ft.	Authority.	Reference.
New Chittravati Bridge, India	Cast iron cylinders in sand, clay, sand and clay, clay and boulders	17 to 64	2·13	E. W. Stoney	P.I.C.E. Vol. CIII. p. 150
Ditto	Ditto	10 to 27	0·85	,,	P.I.C.E. Vol. CIII. p. 150
Ditto	Ditto	44 to 63	2·71	,,	P.I.C.E. Vol. CIII. p. 150
Alloa Bridge, Cal. Rly.	Cast iron cylinders in very mixed strata	74	2·37	C. P. Hogg	P.I.C.E. Vol. CIII. pp. 190, 191
Dalmarnock Bridge, Glasgow	Wrought iron oblong caissons in fine muddy clay, and sandy mud	38 to 53	3·73	,,	P.I.C.E. Vol. CIII. p. 191.
Memphis Bridge	Timber caissons in sand, etc.	40	3·57 (400 lb.—assumed for calculations)	G. S. Morison	P.I.C.E. Vol. CXIV. p. 301

Shoreham Viaduct	Cast iron cylinders sunk by divers, in sand, gravel, and flints	54	2·5	A. S. Biggart	Trans. Inst. Eng. and Ship., Vol. LII. p. 131
Barrow and Suir Viaducts	Steel cylinders in muddy sand	90	1·4	,,	Trans. Inst. Eng. and Ship., Vol. LII. p. 131
Curzon Bridge, India	Double octagonal brick wells sunk by open dredging in fine white micaceous sand	80	2·51 (called sinking effort)	R. R. Gales	P.I.C.E. Vol. CLXXIV. p. 27
,, ,, ,,	Double octagonal stone wells sunk similarly in the same material	80	3·26 (as last)	,,	P.I.C.E. Vol. CLXXIV. p. 27
New Papaghni Bridge, India	Cast iron cylinders sunk in sand, black clay, and silt by open dredging	60	2·08 to 5·6 (called resistance to sinking)	H. J. Thompson	P.I.C.E. Vol. CXXII. p. 189

TABLE III.—PROPERTIES OF MATERIALS.

Material.	Average Weight in Pounds per Cubic Foot.	Ultimate Strength in Tons per square inch.				Modulus of Elasticity in Tons per sq. in. (E).
		Tensile.	Compressive. (On small cubes.)	Shearing.		
Cast iron	450	7 to 11	35 to 60	8	13	6000 to 10,000
Wrought iron	480	21 ,, 24	—	—	—	11,000 ,, 13,000
Mild steel plates and shapes	490	28 ,, 32	—	—	—	11,600 ,, 14,300
Mild steel rivets	490	26 ,, 30	—	—	—	—
				With Grain.	Across Grain.	(Approx.)
Elm	34 to 36	2·7 to 6·3	2·6 to 4·6	—	0·5	400 to 600
Greenheart	60 to 70	—	5·8 ,, 6·8	—	—	500 ,, 600
Jarrah	60 to 63	—	3·2	—	—	—
Karri	63	—	—	—	—	—
Oak, English	48 to 60	4·0 to 6·0	2·0 to 5·0	1·0	2·0	540 to 780
,, American white	50	av. 6·0	av. 3·8	—	—	670
Pine, red (Norway)	32	,, 3·5	,, 2·2	—	—	500
,, white	27	—	—	·11 to ·22	1·1	—
,, yellow	32	av. 3·1	av. 2·2	·2	1·8	690

Pine pitch (Long leaf)	40 to 50	av. 5·3	av. 3·1	·3	2·2	670
,, Oregon (Douglas spruce or fir)	32	,, 3·5	,, 2·5	·2	—	625
,, larch	34	4·0 to 6·3	,, 2·5	·43 to ·76	—	400 to 610
Teak	55	av. 3·6	2·3 to 5·4	—	—	1070 ,, 1250
		Average Crushing Load in tons per sq. foot.				
Sandstone, Corsehill	130	445	See T. H. Beare, P.I.C.E. Vol. CVII. pp. 341 *et seq.*			
,, Polmaise	142	552				
,, Craigleith	139	862				
,, Bramley Fall	132	238				
,, Darley Top	139	517				
,, Arbroath	151	558				
Limestone, Ancaster	156	553				
,, Portland Whitbed	132	205	—			
,, Somersetsh. (av.)	130	105	Safe bearing load may be taken at $\frac{1}{10}$th.			
Granite, Kemnay	163	1110				
,, Corremuir	161	1276				
,, Peterhead	158	1208				
,, Rubislaw	164	1194				
,, Cornish Gray	162	960	—			

Material.	Average Weight in pounds per cubic foot.	Average Crushing Load in tons per sq. ft. [As previous part of Table.]	Safe Bearing Load.			
Portland cement concrete 1 : 8	130	50	6			
,, ,, ,, 1 : 6	to	80	10			
,, ,, ,, 1 : 4	150	100	12			
Ashlar masonry in cement mortar—sandstone	145	150	18			
—limestone	140	100	12			
—granite	165	400	50			
Blue brickwork in cement mortar	135	100	12			
Hard red brickwork in cement mortar	120	60	7			

TABLE IV.—SUPPORTING POWER OF PILES.

Author or Name of Formula.	Formulæ.	Factor of Safety = *fs*.	Terms.
Sanders . .	$F = \frac{Wh}{d}$	8 to 3	F = extreme load in cwts., or the resistance to further penetration
Rankine . .	$F = \sqrt{\left(\frac{4EsWH}{l} + \frac{4E^2s^2D^2}{l^2}\right)} - \frac{2EsD}{l}$	5 to 1	F_1 = safe load W = weight of hammer in cwts.
Trautwine . .	$F = \frac{\cdot 023\sqrt[3]{HW}}{d+1}$	12 to 2	w = weight of pile in cwts. h = fall of hammer in inches H = fall of hammer in feet
Rutter's, or Dutch	$F = \frac{W^2h}{d\,(W+w)} + W + w$	10	d = average penetration, or set, of pile for the last blows in inches D = average penetration, or set, of pile for the last blows in feet l = length of pile in feet
Engineering News	$F = \frac{2Wh}{d+1}$	12	s = sectional area of pile in square inches
Weisbach . .	$F = \frac{W^2h}{d\,(W+w)}$	8 to 3	E = modulus of elasticity of pile $F_1 = F \div fs$

The lower *fs* would be taken in ordinary cases, and the higher where there is much vibration and variation of load. Between the extremes the *fs* could be varied according to the conditions.

TABLE V.—STEEL SHEET PILING.

Name.	Size.	Weight per Square Foot.	Weight per Lineal Foot.	Sectional Area of Pile.	Modulus of Section per Foot.	Radius of Gyration.
				sq. in.	cub. in.	
Universal joist	15 in. × 5 in.	43·00	55·0	16·19 ,,	8·30 ,,	1·42 in.
	10 in. × 5 in.	52·60	45·5	13·37 ,,	11·32 ,,	1·50 ,,
	6 in. × 5 in.	76·20	40·5	11·90 ,,	18·00 ,,	1·57 ,,
Universal joist compound	15 in. × 12 in.	68·78	87·0	25·60 ,,	37·90 ,,	4·34 ,,
	12 in. × 10 in.	76·24	77·5	22·78 ,,	38·17 ,,	3·76 ,,
	10 in. × 6 in.	85·84	70·5	20·72 ,,	28·00 ,,	2·43 ,,
Lackawanna straight web	7 in. × ¼ in.	21·50	12·54	3·7 ,,	·972 ,,	·37 ,,
	12¾ in. × ⅜ in.	35·00	37·1	10·9 ,,	3·756 ,,	·76 ,,
	12¾ in. × ½ in.	40·00	42·5	12·5 ,,	3·876 ,,	·73 ,,
Lackawanna arched web	14 in. × 3$\frac{11}{16}$ in. × ⅜ in.	35·00	40·8	12·0 ,,	6·516 ,,	1·15 ,,
	15 in. × 4⅛ in. × $\frac{7}{16}$ in.	48·00	60·0	17·65 ,,	9·432 ,,	1·29 ,,
Side groove	21¾ in.	42·50	77·0	22·65 ,,	5·30 ,,	1·38 ,,
Ransome—Type D standard section	14¼ in. × 3⅛ in.	28·00	33·0	9·7 ,,	5·945 ,,	1·06 ,,
,, ,, heavy ,,	14¾ in. × 6 in.	44·00	52·0	15·3 ,,	17·90 ,,	2·18 ,,
United States M 102	12 in.	40·00	40·0	11·63 ,,	4·0 ,,	·79 ,,
M 103	9 in.	21·00	16·0	4·71 ,,	1·51 ,,	·56 ,,
M 100	6 in.	22·00	11·0	3·27 ,,	1·48 ,,	·48 ,,
Rothe Erde No. 1	15 in. × 5⅛ in.	22·20	27·7	8·15 ,,	9·06 ,,	1·89 ,,
No. 2	15 in. × 6$\frac{11}{16}$ in.	27·87	34·7	10·2 ,,	13·83 ,,	2·39 ,,
No. 3	15 in. × 7½ in.	38·4	47·97	14·1 ,,	22·37 ,,	2·73 ,,
Wemlinger Type B	12 in. × 2½ × $\frac{3}{16}$ in.	16·5	—	—	1·68 ,,	·88 ,,
Type D	14 in. × 4 in. × $\frac{3}{16}$ in.	23·5	—	—	5·64 ,,	1·41 ,,

TABLE VI.—PIER NO. 2, AMERICAN RIVER BRIDGE, 1910.

	Material Cost. (Dollars.)	Labour Cost.	Total Cost.	Unit Cost.	Unit.	Pier Cost.
Cofferdam or caisson.						
Concrete part. 564 cub. yds.		1259·05	2905·04	5·374	Cub. yd.	
Lumber 3390 B.M.	46·02					
Iron	16·36					
Rail 671 lin. ft.	124·81					
Cement 706 barrels	917·80					
Gravel 780 cub. yds.	271·00					
Fuel and unloading	270·00					
Timber part		2928·60	4486·45	68·432		Labour. 9154·39 4094·10
Lumber 68,412 B.M.	873·93					
Iron	223·92					
Fuel and unloading	460·00					
Pumping and excavating 3325 cub. yds.		2909·44	3950·81	1·239	Cub. yd.	Material. 4265·21 6362·91
Lumber 840 B.M.	14·94					
Sacks	441·50					
Powder	24·93					
Fuel and unloading	560·00					
Concrete 2314 cub. yds.				4·551	Cub. yd.	
Forms						
Lumber 9036 B.M.	147·80	480·37	684·07			

Iron		25·90					Cofferdam and Excavation Pier Total cost $23,876·61
Fuel and unloading		30·00					
Placing			2723·73	8209·94			
Cement	3099 barrels	4028·73					
Gravel	2674 cub. yds.	1011·00					
Iron		12·30					
Rail	492 lin. ft.	49·18					
Fuel and unloading		385·00					
Cars and bunkers			546·00	1189·00			
Lumber	21,580 B.M.	362·00					
Iron		241·00					
Fuel and unloading		40·00					
Backfilling			1457·30	1457·30			
Superintendence and cleaning up		50·00	944·00	994·00			

TABLE VII.

TABLE OF LENGTHS OF SHIFTS AND DECOMPRESSION—PERIODS SUGGESTED AS SAFE BY DRS. LEONARD HILL AND MACLEOD.

Pounds per sq. in. Above Atmosphere.	Shift.	Decompression—Period.
30	4 hours	30 mins. to 1 hr.
45-60	4 hours	1 to 2 hours
75	1 hour	1 to 2 hours
90-105	30 mins. to 1 hr.	2 hours

TABLE VIII.—SHOWING RATE OF DECOMPRESSION IN CAISSON AND TUNNEL WORK.

Working Pressure in Pounds per Square Inch.	Number of Minutes for each Pound of Decompression after the First Rapid Stage.		
	After First Three Hours' Exposure.	After Second or Third Three Hours' Exposure, Showing an Interval for a Meal.	After Six Hours or more of Continuous Exposure.
18-20	2	3	5
21-24	3	5	7
25-29	5	7	8
30-34	6	7	9
35-39	7	8	9
40-45	7	8	9

Illustration.—Pressure 23 lb. by gauge, or 23 + 15 = 38 lb. absolute. Pressure in lock may be reduced in 3 min. to 38 ÷ 2 = 19 lb., or 4 lb. by gauge. Pressure then allowed to fall either uniformly or by about 1 lb. at a time in 4 × 3 = 12 min. Total time = 3 + 12 or 15 min. after first spell of three hours' work. After a second or third spell = 23 min. After a continuous exposure all day = 31 min. Similarly for 40 lb. by gauge, the times would be 91 min., 103 min., and 115½ min.

TABLE IX.

PROPORTION OF SICKNESS AMONG MEN OF DIFFERENT AGES AT BLACKWALL TUNNEL (DR. E. H. SNELL). PRESSURE ABOUT + 35 LB. PER SQUARE INCH.

Men's Ages.	No. of Men Examined and Passed.	Cases of Sickness.	Cases per cent.
15 to 20	55	0	0
20 ,, 25	145	15	10·3
25 ,, 30	152	37	24·3
30 ,, 35	91	19	20·9
35 ,, 40	61	14	22·9
40 ,, 45	38	10	26·3
45 ,, 50	3	5	166·0

DESCRIPTION OF FIG. 5, p. 26.

PRESSURE AT TOP OF CONICAL PORTION AT A, A.—Case 1.

Overturning load, from 56 lbs. wind—		
Wind on superstructure	21·4 tons × 38·6 ft. =	827 ft. tons.
Wind on train	18·0 tons × 43·5 ft. =	783 ,,
Current, 6 ft. per second—		
Pressure	1·85 V^2 = 1·85 × 6^2 = 66·6 lbs. per sq. ft.	
Load	No. 2 piers × 30 ft. × 7 ft. × ·5 × 66·6 = 6·3 tons.	
Current	6·3 tons × 20·0 ft. =	126 ,,
	Total	1736 ft. tons.
Vertical pressure on leeward pier due to overturning loads, 1736 ft. tons ÷ 32 ft. centres of piers		= + 55 tons.
Vertical loads on leeward pier from—		
Superstructure, dead load + live load		+ 250 tons.
Pier, steel work,	14 tons	+ 80 ,,
,, concrete,	66 ,,	
	Total	+ 330 tons.
Total load on one pier at top of conical portion—		
From overturning loads		55 tons.
From vertical loads		330 ,,
	Grand total	385 tons.
Resulting pressure on concrete—		
Area of 7 ft. diameter cylinder		= 38·5 sq. ft.
Resulting pressure, + 385 tons ÷ 38·5 sq. ft		= + 10 tons per sq. ft.

PRESSURE AT BASE OF PIER AT B, B.—Case 2.

Overturning load, from 56 lbs. wind—		
Wind on superstructure	21·4 tons × 51·6 ft.	= 1105 ft. tons.
Wind on train	18·0 tons × 56·5 ft.	= 1017 ,,
Current, 6 ft. per second—		
Pressure	$1{\cdot}85\ V^2 = 1{\cdot}85 \times 6^2 = 66{\cdot}6$ lbs. per sq. ft.	
Load	No. 2 piers × 37 ft. × 7 ft. × ·5 × 66·6 lbs. = 77 tons.	
Current	7·7 tons × 25·0 ft.	= 193 ,,
	Total	2315 ft. tons.
Vertical pressure on leeward pier due to overturning loads, 2315 ft. tons ÷ 32 ft. centres of piers		= + 73 tons.
Vertical loads on leeward pier from—		
Superstructure, dead load + live		= + 250 tons.
Pier, steelwork,	18 tons	= + 129 ,,
,, concrete,	111 ,,	
Water on conical portion, 25 sq. ft. × 20 ft. head		= + 14 ,,
	Total	+ 393 tons.
Displacement of pier, 1393 cub. ft. = 40 tons.		
Total load on one pier at base—		
From overturning loads		73 tons.
From vertical loads		393 ,,
	Total	466 tons.
Deduct displacement at low water		40 ,,
	Grand total	+ 426 tons.
Resulting pressure on bed of river—		
Area of $9'\text{-}1\frac{1}{2}''$ diam. cylinder		65·4 sq. ft.
Resulting pressure, + 426 tons ÷ 65·4 sq. ft.		= + 6·5 tons per sq. ft.

INDEX.

A

Abutment foundations, 12-16.
Air, compressed, amount required, 99, 111.
— — effects of, 102-13.
— — tables, 133, 134.
Air-locks, 84, 85, 99, 100, 101.

B

Blasting in caissons, 96.
Bores, general observations, 5-7.
— preliminary to design, 5.
— use with caution, 6.
Braking, effects of, 22, 23.
Buoyancy, 17, 24, 25.
Buoyant foundations, 114.

C

Caisson disease, 103 *et seq.*
Caissons, open, 76-83.
— — American River Bridge, 82, 83.
— — Cannon Street Bridge, 77.
— — Charing Cross Bridge, 77.
— — Hawkesbury Bridge, 76.
— — Poughkeepsie Bridge, 76, 80, 81.
— — Tower Bridge, 77.
— pneumatic, 84-101.
— — design, 90.
— — excavation in, 95, 96.
— — plant, 99-101.
— — righting of, 96-9.
— — sinking, 95-7.
Caissons, pneumatic, Barrow Bridge, 88.
— — Carmarthen Bridge, 88.
— — Eads's St. Louis Bridge, 86.
— — bridge, Kansas City, 88-90.
— — King Edward Bridge, 98.
— — Quebec Bridge, 98.
— stranded, 67-75.
— — French bridges, 75.
— — Knipplesbro Bridge, 72, 75.
— — Langebro Bridge, 72, 74.
— — Walnut Street Bridge, Green Bay, 69-72.
Cast iron cylinders, 78.
— — — Clyde Bridge, 78.
Clay, 9.
Compressed air, effects of, 102-13.
— — Tables, 133, 134.
Compressol piles, Risorgimento Bridge, 117, 118.
Concrete caissons, 82, 83.
— piles, 32, 117, 118.
— strength of, 16, 128.
Current on piers, 21, 22.
Cylinders, cast iron, 78.
— pier, 26, 57, 78.
— screw, 51-8.
— steel, 77.

D

Dams, 34-50.
— depths of, 34, 35.
— loads on, 37
— steel piling, 36, 43-50.

Dams, timber, 40-3.
— types, 34, 35.
Deep foundations, Barrow Bridge, 88.
— — Eads's St. Louis Bridge, 86.
— — Hawkesbury Bridge, 76.
— — Poughkeepsie Bridge, 76.
— — Sara Bridge, 76.

E

Excavation in caissons, 95.

F

Foundation bed, allowable net pressures, 123.
— — depth of, 10-11.
— — supporting power, 5-10.
— — testing, 5.
— Company, work by, 118-21.
Foundations, abutment, 12-16.
— buoyant, 114.
— pontoon, 115.
Friction, skin, 17, 24.
— — tables, 124, 125.

G

Gravel, 9.

I

Ice, 21, 22.
Impact, 19.

L

Lackawanna steel piling, 44-7.
— — — Lake Shore Bridge, 45.
— — — 92nd Street Bridge, Chicago, 46, 47.
Lateral forces, 17.
Loads, allowable foundation, 123.
— dead, 17, 18.
— live, 17, 19.
Loads, wind, 17, 19.
— on dams, 37, 38.

M

Materials, properties of, 126-8.

P

Pier foundations, 17-26.
— — loads, 17.
— — reactions, 17.
Piers, cylinder, 26.
Piled foundations, Avon Swing Bridge, 28.
— — Bonn Bridge, 116.
— — Breydon Viaduct, 28.
— — Walnut Street Bridge, Green Bay, 32.
— — Waterford Bridge, 28, 32, 33.
Piles, bearing, 27.
— disc, 52.
— driving of, 30.
— — tests, 28.
— methods of use, 31.
— sheet, 27, 34, 40.
— steel interlocking, 27, 36, 43-9.
Placing caissons, 93, 94.
Pontoon foundations, Constantinople Bridge, 114.
— — Howrah Bridge, 214.
— — Spencer Dock Bridges, Dublin, 116.
— — bridges over Weaver, Northwich, 115, 116.

R

Ransome steel piling, 44, 49.
— — — Kingston Bridge, 49.
— — — Warrington Bridge, 49.
Reactions, pier, 17.
Righting caissons, 96-8.
Rock, 10.
Rothe Erde steel piling, 44, 49.

S

Sand, 9.
Scour, 11.
— protection against, 118.
Screw piles and cylinders, 51-8.
— cylinders, Chiromo Bridge, 54-8.
— — Usk Viaduct, G.W. Railway, 54.
— piles, Barrowfield Iron Works, 54.
Screwing piles, 53.
— — assisted by water jet, 53, 54.
Settlement of foundations, 7-9.
Side groove steel piling, 44, 47, 48.
— — — — Southwark Bridge, 47.
Simplex piles, Beaver Bridge, 117.
Steel caissons, 72, 77-80, 90-3.

T

Testing foundation beds, 5.
Timber caissons, 80-1, 88-90.
— kinds in use, 29.
Timber caissons, strength of, 40, 126, 127.

U

United States steel piling, 44, 49.
— — — — Sara Bridge, 49.
Universal joist piling, 44, 45.
— — — Deptford Bridge, 45.
— — — Kelvin Bridge, 45.

W

Water jet, 53, 54.
Well foundations, 59-66.
— — in Egypt,60.
— — in India, 59, 60.
— — sinking of, 61.
Wells, brick, Cairo Bridge, 64.
— — bridges, Central Argentine Railway, 64, 66.
— — — Pegu-Moulmein Railway, 63.
— — Empress Bridge, 62, 63.
— — Esk Viaduct, N.E. Railway, 64.
— masonry, Koyahkai Bridge, 63.

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